OHIO STATE UNIVERSITY STUDIES

ENGINEERING EXPERIMENT STATION BULLETINS NUMBER 37

GUNITE AND CONCRETE ENCASEMENT TO INCREASE THE STRENGTH OF STRUCTURAL STEEL

BY

CLYDE T. MORRIS

AND

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THE ENGINEERING EXPERIMENT STATION

Price Fifty Cents

1928

THE OHIO STATE UNIVERSITY

COLUMBUS

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THE OHIO STATE UNIVERSITY
THE ENGINEERING EXPERIMENT STATION

BULLETIN NO. 37 1928

GUNITE AND CONCRETE ENCASEMENT TO INCREASE THE STRENGTH OF STRUCTURAL STEEL

BY

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FOREWORD

ENCASEMENT OF STEEL AS A MEANS OF ELIMINATING WASTE

A satisfactory way to keep bridges and buildings longer in service is a conservation measure, an elimination of waste. The tests reported in this bulletin indicate that the use of reinforcing steel and gunite encasement is satisfactory, not only in restoring the strength of corroded beams and girders, but also in enabling them to bear heavier loads than those for which they were originally designed.

These tests show that, within reasonable limits, it is possible to reinforce beams to a calculated desired strength. To take just one example: I-Beam No. XVII was removed from the Allegheny Coaling Station as unfit for further service because it had been so eaten away as to have little more than half its original strength. Using an allowable fiber stress of 18,000 pounds per square inch the load-carrying value of this I-beam had been reduced from 127,500 pounds to 79,000 pounds, a loss of 38 per cent. Reinforced and encased this I-beam required a load of 152,500 pounds to produce a unit stress of 18,000 pounds, an increase of 93 per cent over the strength of the corroded beam or of 19½ per cent over that of a new beam.

Mr. C. Clement Cooke speaking before the Engineering Section during the Management Week program at Ohio State University, October 26, 1926, said: "As we now see it, this class of work will be largely confined to the strengthening of existing bridges and viaducts, with perhaps some building work. Vehicular traffic on top of the viaduct will not interfere with the success of the work. Also, several railroad engineers have told me that, so long as it is possible to maintain a bridge it is their policy to do so, rather than to tear it out and rebuild. The present ruling of the Public Service Commission requires that new bridges over railroad traffic shall have a minimum clearance of 21 feet 6 inches. Many bridges and viaducts throughout the country, built before this ruling was made, do not have this clearance. If these structures are torn out the new bridges replacing them must have the required clearance. This requires either the raising of the street grade, which often means the condemnation of valuable property adjacent to the bridge, or the lowering of the railroad tracks, which, in congested sections, is most expensive."

This bulletin is published to make available to engineers and designers the data which have been obtained. It is hoped that it will stimulate further investigation and that practical use will largely be made of the methods described.

E. A. Hitchcock, *Director.*

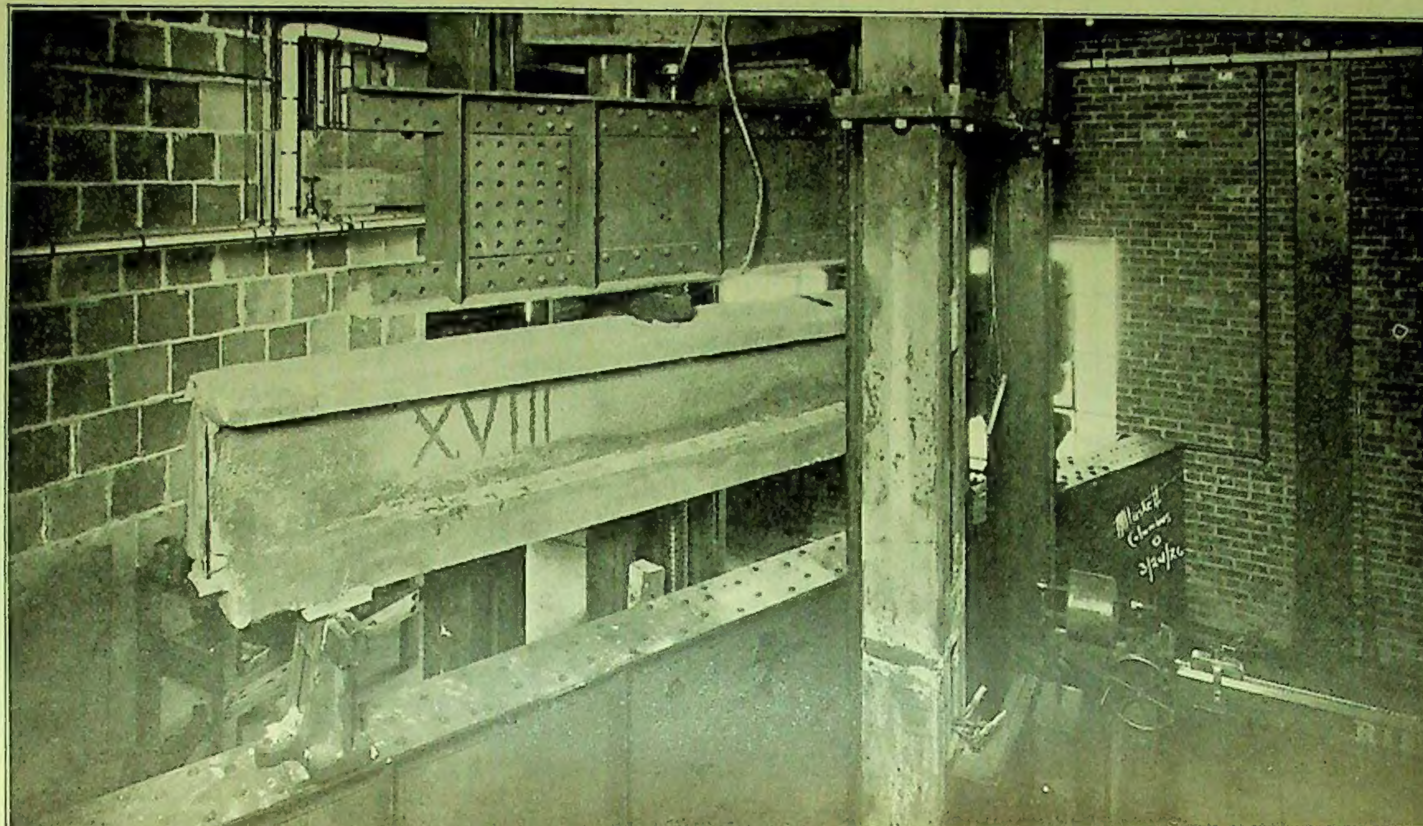


FIGURE 1. FRONTISPIECE, BEAM IN TESTING MACHINE

GUNITE AND CONCRETE ENCASEMENT TO INCREASE THE STRENGTH OF STRUCTURAL STEEL

Synopsis: Both gunite and concrete encasement have been used for a number of years for the purpose of fireproofing structural steel members as well as protecting them from corrosion. Heretofore little consideration has been given in design to the strength which is added to the member by means of the protective encasement; in fact, standard practice is to penalize the member for the additional dead load of the encasement. Nor has much thought been given to the possibilities of reinforced encasement to strengthen, in place in existing structures, steel members which may have been considerably weakened by corrosion, or which, after years of service, may be of inadequate section to carry loadings in excess of those for which they were originally designed.

Experiments made in England and Canada have indicated that, within reasonable limits, concrete encasement will act as a unit with the member in supporting loads. Tests at the Engineering Experiment Station of Ohio State University have borne out these conclusions, have demonstrated that badly corroded steel members may be restored to original strength by including in the encasement properly designed reinforcing steel, and have shown that encasement materially increases the stiffness of steel columns. At the North High Street Viaduct in Columbus, Ohio, restoration of reduced steel girders by reinforcing steel and gunite has received a practical test and has been found satisfactory.

There are many steel structures whose terms of usefulness have apparently expired, either through corrosion or because of the ever-increasing live loads which are being put on them. The deterioration of steel structures over railroad crossings is especially rapid on account of the corrosive action of locomotive gases; and the blast from the stacks quickly removes any paint or similar protective coatings which may be used to prevent access of gases and weather to the steel. Industrial buildings also are oftentimes subject to similar conditions.

C. Clement Cooke, of the Fritz-Rumer-Cooke Company, Columbus, Ohio, conceived the idea that the strength of any steel girder which had become reduced in section might be restored, at least in part, by adding sufficient reinforcing bars to replace the metal which had corroded and bonding these bars to the old girder and completely encasing the reinforcing bars and old member with

cement mortar applied by means of a cement gun. Mr. Cooke consulted the writers of this bulletin, J. F. Leonard, Engineer of Bridges and Buildings, Pennsylvania Railroad System, Lines West, B. C. Collier, President of the Cement Gun Company, Inc., and others. The method suggested was considered well worthy of experimentation, and the problem of developing a satisfactory design for strengthening test members was presented. It was considered essential that the method developed be entirely applicable to actual work in the field. A design was made by Walter Braun, Consulting Engineer, of Columbus, Ohio, on the assumption that the member to be strengthened might be a girder, beam, or stringer in a floor system supporting the reinforced concrete slab of a bridge or building.

The practicability of Mr. Cooke's idea was demonstrated in a series of tests made at the Engineering Experiment Station of the Ohio State University in the spring and summer of 1926. In the summer of 1927 the method was tried on a structure in actual use, the North High Street Viaduct at Columbus, Ohio, with highly satisfactory results.

The great advantage of this method of repair is that it can be done with the cement gun without disturbing the roadway or floor slab or seriously interfering with traffic.

In addition to the experiments with corroded beams at the Engineering Experiment Station, comparative tests were made of gunite and poured concrete as encasement for new beams and columns. From the results observed in all the tests the following conclusions have been drawn:

1. The bond of gunite encasement and of poured concrete to old or new steel members is sufficient to insure their action as a unit under loads.

2. The strength of steel beams and girders can be increased, within reasonable limits, by the addition of steel reinforcing rods and gunite or poured concrete encasements.

3. The strength of such reinforced beams may be predicted accurately by the usual theory of composite beam action.

4. Steel beams or columns encased in gunite or poured concrete for fire protection may be calculated as composite members and the strength of the encasement utilized as assisting in carrying the loads. In making use of this additional strength it must be remembered that the encasement is primarily for fire protection, and that a prolonged and hot fire may injure the outer surface of the encasement to a certain depth. Plasticity or the tendency of concrete to flow under stress also must not be forgotten.

5. The value to assume in design for the ratio of the moduli of elasticity depends upon the gunite or concrete used. Though there is reason to believe that gunite when properly tested will show considerably higher strength than poured concrete, the tests indicate that the modulus of elasticity does not

vary in this manner. Throughout the tests a value of 10 for the ratio of the moduli of elasticity seemed too low. Much investigation remains to be done as the theory rests on this ratio and the proper allowance for plastic flow can be expressed in it.

6. The stiffening effect of any encasement is of considerable value and may be given due consideration in designing.

7. Care in curing gunite under ordinary conditions after seven days is hardly justified economically.

8. Gunite adheres better than poured concrete to structural steel and is therefore a better form of protection from corrosion.

9. The relation of strength to modulus of elasticity of ordinary poured concrete cannot be used to determine the modulus of elasticity of gunite from its strength.

THEORY OF REINFORCEMENT

The correct theory of the composite action of concrete-encased steel beams was first published in 1912 by Professor Ewart S. Andrews of Goldsmith's College, London. Various handbooks and tables have been published since that time in England giving the theoretical capacities of steel I-beams embedded in concrete.

FIGURE 2. BEAMS READY FOR ENCASEMENT

The theory of the composite action of beams composed of two materials differing in elastic properties is well known and will be found in any standard textbook on strength of materials. It is illustrated by the old style flitched beam consisting of a steel plate tightly bolted between two wood beams. As long as the friction

developed by the tension on the bolts is sufficient to cause the two materials to deform equally the theory is correct. Application to a steel beam covered with concrete* follows directly. The composite beam theory holds so long as the bond between the steel and the concrete is unbroken. The added factor of including reinforcing bars in the concrete encasement does not alter the theory, but merely complicates its application to a certain extent.

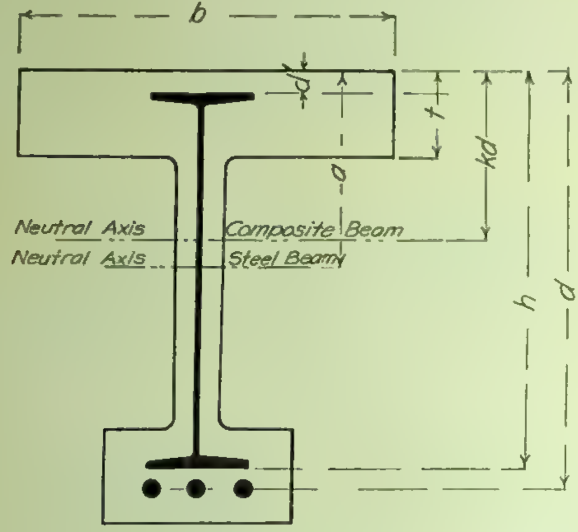


FIGURE 3

In the computations of the properties and probable strengths of the beams tested, use was made of what is known as the transformed-section method, considering the steel as its equivalent area of concrete, that is, multiplying the steel area by the ratio of the modulus of elasticity of steel to that of concrete. All steel, old and new, was considered as working both in tension and in compression. Calculations were made on the assumption that the concrete would act in compression only. Figure 3 shows a section of such a composite beam. The symbols used and an illustrative problem are given in the first section of the Appendix.

PREVIOUS TESTS OF BEAMS

Professor C. R. Young of the University of Toronto in "Haunching as a Reinforcement for Steel Beams" published in the *Canadian*

* Concrete is here used in a general sense to denote both gunite and placed or poured concrete.

Engineer for December 22, 1925, gives a review of previous investigation of the strengthening effect of concrete encasement.

The first tests of record were made at the National Physical Laboratory, Teddington, England, in 1922-23.[1] These tests were of slabs whose reinforcement consisted of steel I-beams embedded in the concrete. The slabs varied in thickness from 4 in. to 10 in., and the I-beams from 4 in. to 6 in. in depth. It was noted that the concrete slabs were much stiffer than the plain joists, and that notwithstanding the low crushing strength of the concrete it was sufficient to develop the limit of elasticity in the steel. The general conclusions were: "These tests indicate clearly a large gain of strength in rolled steel joists embedded in concrete when the top of the concrete is raised above the top flange of the steel. Little or no gain of strength is obtained when the concrete is flush top and bottom with the flanges of the joists. For every inch of concrete above the top flange the gain in strength over the plain joists is considerable; and this gain of strength continues to increase at a somewhat higher rate than the increase in thickness of the concrete above the top flange."

In the winter of 1922-23 a very complete series of tests was run on two floor panels at the plant of the Dominion Bridge Company at Lachine by Professors H. M. MacKay of McGill University, Peter Gillespie of the University of Toronto, and C. Leluau of the University of Montreal.[2] Each panel consisted of a reinforced concrete slab 10 x 16 ft. and 4 in. thick, reinforced by wire mesh. The slab was supported by two 10-in. I-beams, 22¼ lb., 16 ft. long, spaced 5 ft. center to center. The top flanges projected 1 in. into the concrete slab and the concrete encasement projected 2 in. below the bottom of the lower flange. The encasement varied in width from 9½ in. at the bottom to 11½ in. at its junction with the slab. It is seen that this follows closely the usual practice in encasing beams for purposes of fireproofing. From these tests the observers concluded:

1. That in the type of construction considered the concrete and steel act together to form a composite beam.

2. That within practical limits the composite beam conforms to theory and may be designed by methods analogous to those used in the case of reinforced T-beams.

3. That a considerable economy in the support of live loads may be obtained by utilizing the joint action of the concrete and steel.

[1] "Tests of Steel Beams in Concrete at the National Physical Laboratory," Redpath, Brown & Co., Ltd., 1924.

[2] *Engineering Journal,* August, 1923.

4. That a further economy can probably be obtained by modifying the construction so as to provide for a measure of continuity at the ends of the beams.

5. That the absence of stress on the concrete, and of bond stress due to the dead load, is advantageous to the haunched beams as compared with reinforced concrete beams.

6. That the nature of the shear reinforcement provided by the webs of the steel I-beams would probably eliminate failure due to diagonal tension.

7. That the stability and endurance of this form of construction depend upon the bond between the concrete and the steel, more especially at the top flange of the I-beam. The safe limits of the bond stress have not yet been determined.

In 1923 tests to determine the resistance of encased beams to bond or horizontal shear failure were made by Professors Peter Gillespie and R. C. Leslie at the University of Toronto, and published in Bulletin No. 5, "Steel I-beams Haunched in Concrete," of the University of Toronto Faculty of Applied Science. The tests show that the deflection of the composite beam follows the law of the homogeneous beam, and appear to warrant the following assumptions:

1. The two materials act together as they do in reinforced concrete.

2. Live load deflections follow with necessary changes approximately the formulae for homogeneous beams.

3. A working stress of 240 lb. per square inch in horizontal shear is not excessive.

A series of similar tests was made in 1924 by Dean MacKay of McGill University who says in his report:

"As it was thought advisable to try the effect of vibration, a concrete mixer weighing (with its charge of 500 lb. of stone) 10,400 lb. was next placed on slab No. 1 and run for two days at a good speed, compressed air being used in the cylinders. The load was concentrated on a length of 3 ft. 9 in over each beam, giving a bending moment at the center of the beam of 18,400 foot-pounds which corresponds to a uniformly distributed load of 115 lb. per square foot. Howard gauge and deflection readings taken before and after the running of the mixer checked the weight of the latter, but gave no other result. The hair cracks noted showed no further extensions, although the amount of vibration produced would have been quite excessive in a building. On account of the short duration of this test, however, it must be regarded as inconclusive."

The Ohio Tests

The tests made at the Engineering Experiment Station of the Ohio State University were primarily for the purpose of determining whether a corroded steel beam could be reinforced sufficiently to restore it to its original new value. The Fritz-Rumer-Cooke

Company, railroad construction contractors of Columbus, Ohio, prepared the gunite-encased beams and assisted in running the tests. The Pennsylvania Railroad System, through the courtesy of J. F. Leonard, Engineer of Bridges and Buildings, Lines West of Pittsburgh, furnished the old beams.

BEAMS USED

Three classes of beams were tested:

Class I. Corroded Built-Up Beams. Four 21-in. wrought-iron plate-girder stringers from the Adams Street Viaduct in Chicago, erected in 1885. These are referred to as Beams XI, XII, XIII, and XIV.

Class II. Corroded I-Beams. Four 24-in. 80-lb. steel I-beams from the Allegheny Coaling Station at Pittsburgh. These are designated as Beams XV, XVI, XVII, and XVIII.

Class III. New I-Beams. Three new, 22-in. 86.5-lb. Bethlehem I-beams, purchased to determine what strengthening effect could be obtained by encasing new beams in concrete. These members are referred to as Beams A, B, and C.

The plate-girders and old I-beams were badly reduced in section, the strengths of the various members, as calculated from caliper measurements made by the Pennsylvania Railroad, being reduced from 18 per cent to 64 per cent below those of the original sections. Test loads of bare beams exceeded those computed for the calipered sections, indicating that measurements had been taken at sections which showed up particularly bad, a leaning toward the side of safety an the part of the inspector.

The condition of the corroded members may be seen in Figures 4, 19, 20, 21, 32, 33, 34, and 35 made from the caliper measurements.

One member from each of the three groups was tested bare.

One member from each of the three classes was encased with gunite, unreinforced except for 3 x 3 in., No. 8 x No. 8 electrically-welded wire mesh. The steel was cleaned of rust and dirt, the mesh was put in place, and the gunite was built out to a thickness of 1½ to 2 in. Four holes about 8 in. in diameter burned in the web of each member permitted a union of the gunite on both sides of the beam, thus increasing the bond of the gunite to the steel. Figure 5 shows a corroded beam with wire mesh in place. The drawing, Figure 6, is a section showing the mesh held in place by small rods.

With two old stringers and two old I-beams as bases, composite sections were made having concrete T-heads at the top and sufficient steel reinforcing rods in the bottom to supply the strength

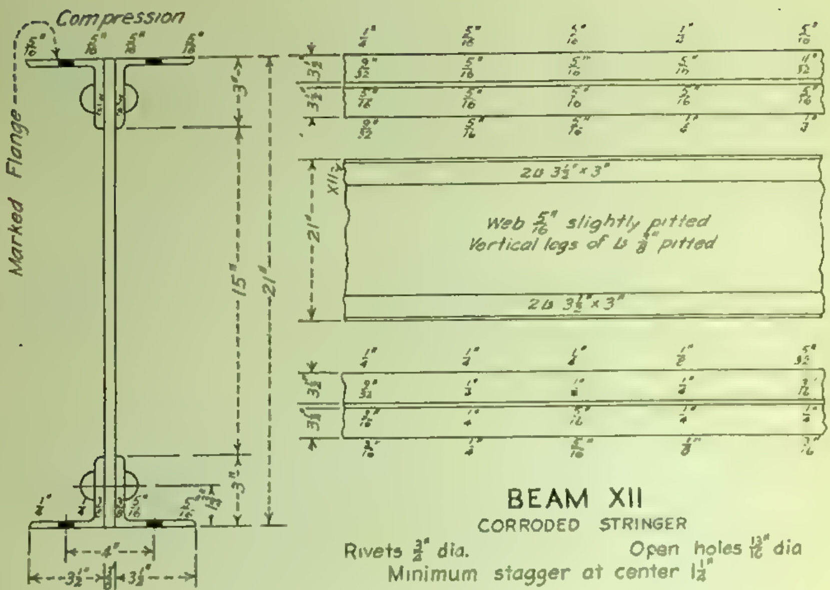


FIGURE 4

lost through the corrosion of the bottom flanges. The reinforcing steel was welded into the web or flange of the corroded members near the ends. Welding seemed to have no positive value, however, and was omitted in the High Street Viaduct tests. One stringer, Number XI, had the bars bent up as for shear reinforcement. Figure 7 shows the composite section of this member. Figure 8 is a photograph of beam Number XVII ready for encasement.

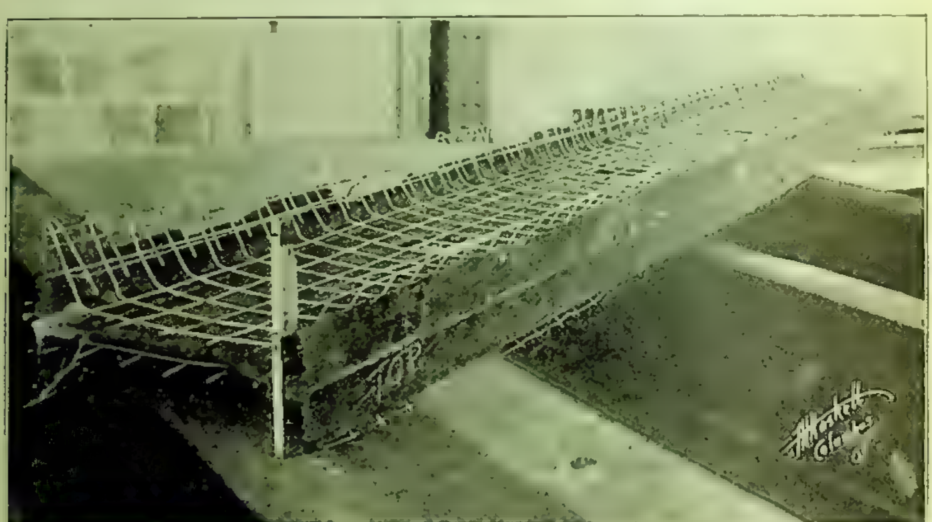
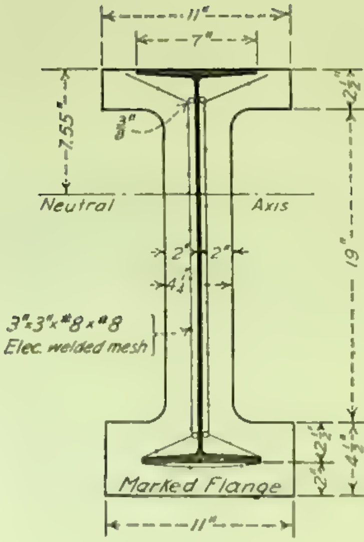
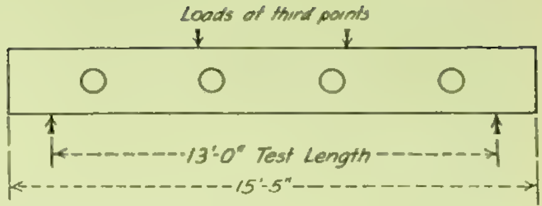


FIGURE 5





BEAM XVI

Gunite applied March 25, 1926. Tested April 1, 1926. 4-8"ϕ holes burned from web in order to bond gunite on either side. Dry Mixture, 1 of cement, 3 of sand by volume. Mixture in place 1:2¼ (about) owing to the usual rebound of sand during shooting. Universal cement and washed local river sand used. For data on steel member see Fig. Air pressure at cement gun 35# to 40#

Moment of Inertia 1107.2 in.^{4}

Section Modulus, Compression 146.4 in.^{3}

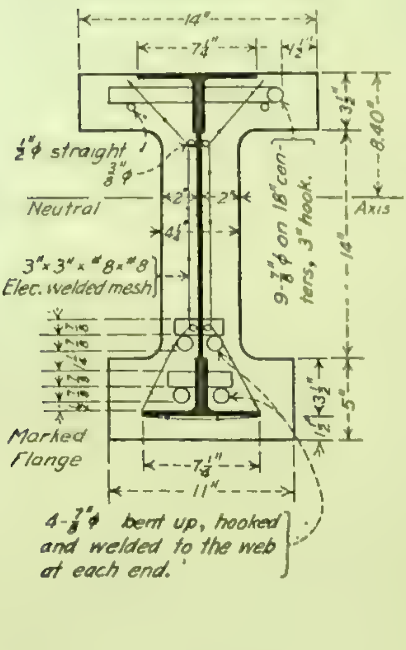
Tension 67.2 in.^{3}

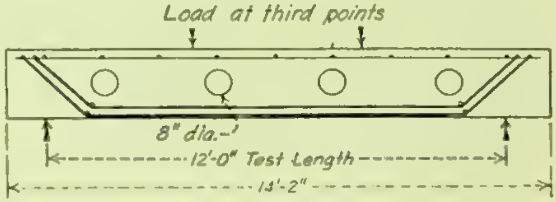
FIGURE 6

Moment of Inertia 1302.7 in.^{4}

Section Modulus, Compression 154.8 in.^{3}

Tension 103.4 in.^{3}





BEAM XI

Gunite applied February 25, 26, 27, 1926.
Tested March 26, 1926.
4-8"ϕ holes burned from web in order to bond gunite on either side.
Dry Mixture, 1 of cement, 3 of sand by volume.
Mixture in place 1 2¼ (about) owing to the usual rebound of sand during shooting.
All reinforcing rods, except ⅜"ϕ furring, are deformed O.H. steel. The 4-⅞"ϕ rods in the bottom flange are bent similar to the usual bent up rods with hooked ends in concrete beams, except hooks are at right angles and are welded in rivet holes in the flange of beam.
For data on steel member see Fig.
Universal Cement and washed local river sand used.
Air pressure at cement gun 35 to 40 pounds.

FIGURE 7

Thus tests were arranged for comparison of beams without encasement; with encasement containing no steel other than the mesh; and with encasement plus reinforcement to restore the reduced sections.

One new I-beam was encased with poured concrete to make practically the same section as that of the gunited new beam.

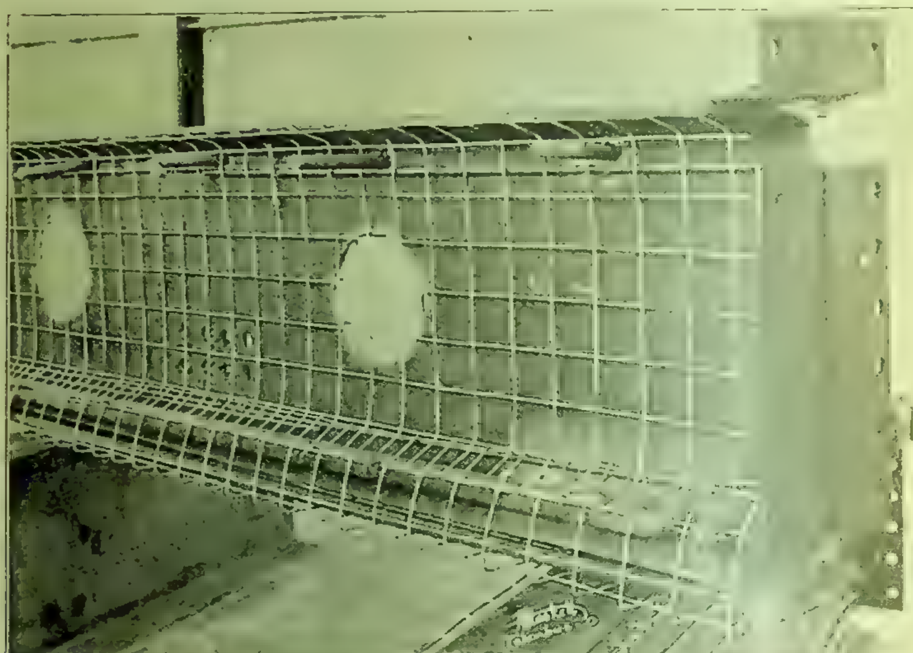


FIGURE 8

without reinforcement other than wire mesh. The object was a comparison of the effect of poured concrete encasement with that of gunite encasement.

PROCEDURE

All tests were made at the Ohio Engineering Experiment Station. Beam C was tested on a 400,000-lb. Riehlé beam testing machine; the remaining pieces were tested on a 500,000-lb. column machine, plate girders being used above and below the test members, as shown in Frontispiece. Fiber strain measurements were made with a 20-in. Berry Strain gage. Deflections were measused to 1/200 in. by direct readings on a scale attached to the middle of the beam, using a fine steel wire stretched from one end of the beam to the other and a small mirror by the scale to avoid parallax. Whitewashing the pieces helped materially in locating and tracing small cracks.

For testing, the beams rested on 8 x 12 x 1 in. plates, bedded the day before the test with a mortar of equal parts portland cement and plaster of paris. Equal loads were applied at the third points through 2-in. rollers resting on 4-in. steel billets which in turn rested on small steel plates bedded to the steel of the beams except in beams B and C for which the plates were bedded directly on the encasement. The rollers and billets provided enough room between the test member and the upper plate girder to permit the use of the strain gage. For the extensometer readings small holes about 1/16 in. deep were made with a Number 56 twist drill and reamed

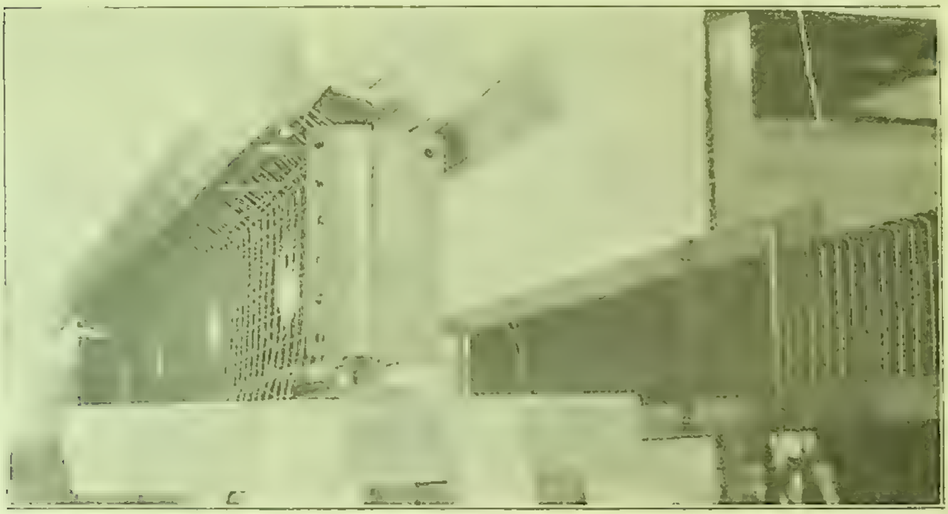


FIGURE 9. REINFORCEMENT, MESH, AND SHOOTING STRIPS IN PLACE

with a center punch to get a surface at the rim which would not appreciably wear during a number of applications. Holes for access to the steel through the encasement were obtained by digging out plaster of paris plugs placed before the gunite was applied. For readings on the encasement the strain gage holes were made in pieces of 3/8-in. rod bedded 3/4 in. deep in the concrete.

After stringer Number XII had been tested a piece was cut from the flange and the modulus of elasticity of the wrought iron found to be 28,000,000 in English units. This value was assumed for all the plate girders. For both old and new I-beams E was taken as 30,000,000 because the theoretical line based on 30,000,000 coincided very closely with the observed data of Beam B.

For details of individual test specimens see page 22.

RESULTS

Table I on page 13 gives in condensed form the results of the tests of these beams. The properties of the sections shown are for the beams as tested, using 10 as the ratio of the modulus of elasticity of steel to that of gunite, and considering all steel, including the mesh, as working. Beam C was encased in poured concrete, and "n" was assumed to be 15. The "Observed Loads Carried at 16,000 lb., per sq. in. on Metal" are taken directly from the stress deformation curves. The "Loads Computed from Section Moduli" are computed from the section moduli shown on this table and are shown graphically on the stress deformation curve sheets as "Theoretical Composite" curves. The remainder of the table is self explanatory, except that a percentage representing ultimate load is given where "Local Failure" is indicated for "Ultimate Load." These percentages are figured for the last and highest load observed just prior to the time of the local failure.

The table does not show the computed load capacity at 16,000 lb. per square inch for the original unreduced plate girders and I-beams. These values for the plate girders are 65,000 lb. in tension and 70,800 lb. in compression and for the I-beams 105,700 lb. for both flanges. All the reinforced encased beams carried loads well over these values, and in only a few cases did they carry less than the loads calculated for the composite beams. All the loads given are superimposed loads (excluding weight of test beams.)

The stiffening effect of the encasement is clearly shown in the amount of load taken above that which produced 16,000 lb. per square inch in the tension steel (see the last column of the table.) A comparison of the results for Beams A and B shows this particularly.

A graphic representation of the increase in loads required for certain deformations and unit fiber stresses is shown in Figures 10 and 11. The base line for each beam is its value if tested in a naked condition computed for the section indicated by the caliper notes.

The North High Street Viaduct Tests

Shortly after the tests at the Engineering Experiment Station had demonstrated the possibility of strengthening corroded steel members, an opportunity for a practical test was presented at the North High Street Viaduct in Columbus, Ohio, where the badly corroded condition of the steel work made necessary either replacement or repair of the structure. As Figure 12 shows, holes in the

TABLE I

Tabulation of Working and Ultimate Load Data

Member		Test Span	Type and Age at Testing Days	Moment of Inertia	Section Moduli		Observed Loads Carried at 16,000 Lbs. per Sq. In. on Metal		Loads Computed from Section Moduli 16,000 Lbs. per Sq. In. on Metal		Difference in % of Computed Load Values		Ultimate Load	In % of Obs. Load at 16,000 per Sq. In. Tension
					Tension	Comp.	Tension	Comp.	Tension	Comp.	Tension	Comp.		
Plate Gdrs.	XII	12′-0″	Bare	890.8	80.4	89.8	60,000	66,500	52,400	58,600	+13	+12	84,300	140
	XIII	12′-0″	Plain-7	975.8	75.8	120.3	65,000	76,000	48,000	77,500	+35	— 2	164,300	253
	XIV	12′-0″	Reinf.-28	1385.5	111.7	161.1	95,000	101,000	72,000	104,700	+32	— 4	225,700	237
	XI	12′-0″	Reinf.-28	1302.7	103.4	154.8	80,000	104,000	66,200	100,200	+21	+ 4	227,950	285
I Beams	XV	13′-0″	Bare	2087.2	79.9	105.2	68,000	85,000	48,100	63,700	+29	+25	146,825	216
	XVI	13′-0″	Plain-7	1107.2	67.2	146.4	73,000	90,000	38,800	87,900	+88	+ 2	Local Failure	228
	XVIII	13′-0″	Reinf.-28	2385.4	176.8	241.5	136,000	135,000	104,900	144,800	+30	— 7	Local Failure	186
	XVII	13′-0″	Reinf.- 7	2453.7	166.8	246.0	129,000	145,000	99,000	147,600	+30	— 2	328,300	254
I Beams Beth.	A	19′-0″	Bare	1629.3	147.7	147.7	62,000	59,000	60,800	60,800	+ 2	— 3	140,850	227
	B	19′-0″	Plain-29	2190.5	157.6	206.3	76,400	——	62,200	——	+23	——	222,050	290
	C	19′-0″	Plain-28	2025.	151.2	190.5	75,000	——	64,000	——	+17	——	202,630	270

webs of some of the girders were large enough for a man to crawl through. It was suggested that encasement of the old girders in reinforced gunite might restore the carrying capacity of the viaduct, add a considerable period of years to its service, and at the same time save money and the inconvenience of detouring traffic during construction.

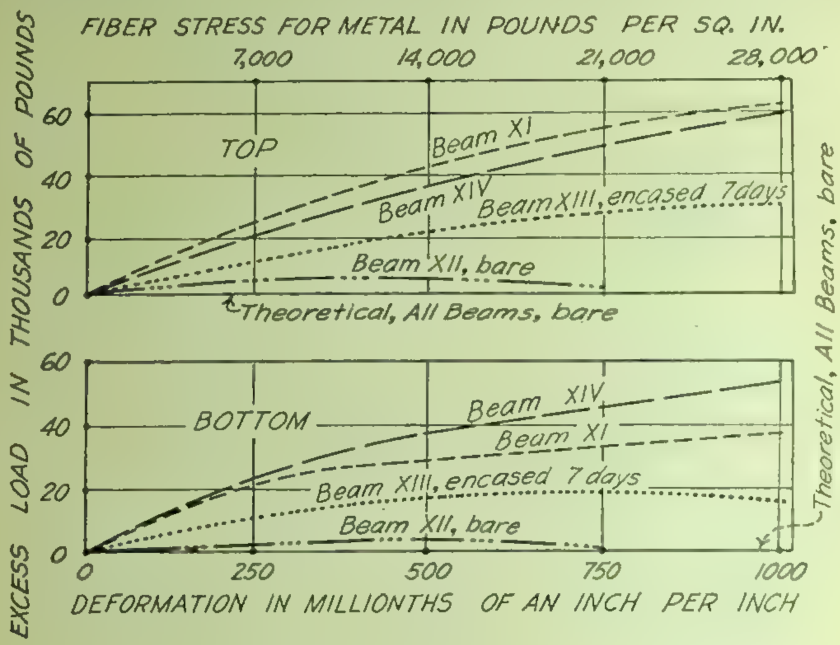


FIGURE 10

Although the results obtained with encased beams and girders at The Ohio State University had been very satisfactory, City Engineer R. H. Simpson decided that this method should not be adopted for the North High Street Viaduct without some further investigation as to its effectiveness when applied to members of much greater span and relative slenderness and under adverse conditions. It was proposed that tests be made on two girders in order to check, if possible, the results obtained on the encased beams tested in the laboratory. This demonstration fulfilled expectations completely. Arrangements have now been made for the encasement and strengthening of about 40 per cent of the girders in the viaduct; others can be repaired as their condition requires it, and finances will permit.

The North High Street Viaduct at the Union Station was built in 1894. It consists of four spans each 76 ft. long, and carries a 65-ft. paved roadway with two sidewalks. The construction con-

sists of parallel longitudinal steel girders about 4 ft. 6 in. centers and 48 in. deep, supporting a buckle-plate floor with concrete filling and asphalt wearing surface. It has a clearance of 16½ ft. above the railway tracks.

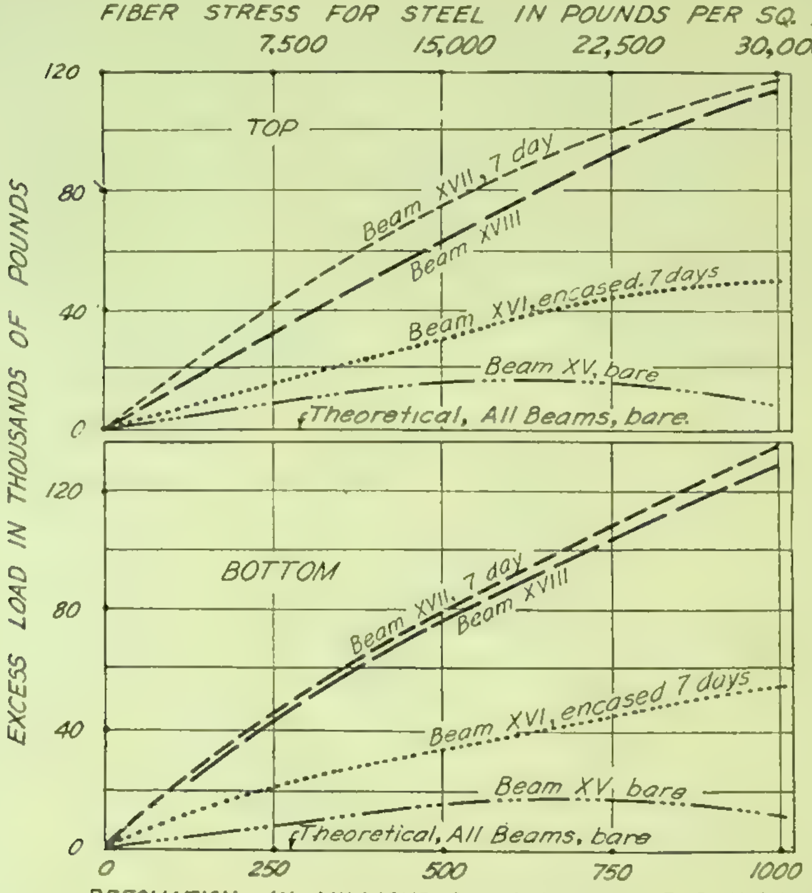


FIGURE 11

Because of the limited clearance, the erosive action of locomotive exhaust and the action of gases on the steel have caused serious deterioration and heavy expense for maintenance. In 1912 a careful investigation showed that the flange plates and the stiffeners of many of the girders were greatly reduced in cross-section, and the webs of some of the girders were rusted through; at that time the badly corroded girders were strengthened by the addition of new flange plates, stiffeners and web plates. Inspection at intervals disclosed further deterioration and indicated that the structure

should be rebuilt, so preliminary studies were made and estimates prepared for its complete renewal. It was thought that a new viaduct would cost $175,000, and would also require the construction of a temporary roadway to the Union Station and the rerouting of traffic on High Street for approximately a year.

FIGURE 12

The two girders selected for the test, Numbers 14 and 15, were not the worst members in the structure, but were badly corroded.

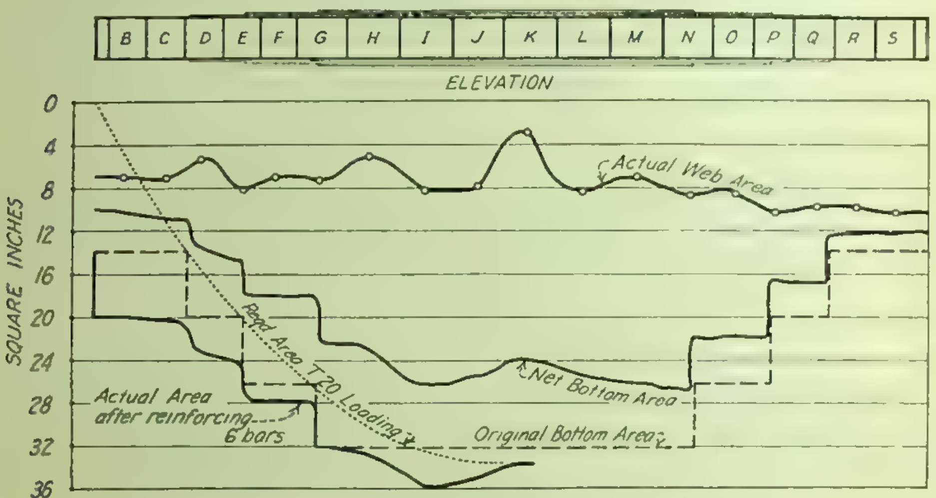


FIGURE 13

The condition of Number 15 is shown in Figure 13. They were chosen because, on account of their location, they could be tested under load without seriously interfering with traffic. Motor trucks were placed in position as shown in Figure 14 over these girders,

and deflection measurements and strain-gage readings of the deformation of the bottom flanges were taken. Caliper measurements were made of the steel sections, the girders were cleaned, reinforcing rods designed to bring the lower flange steel up to the necessary section were added, and the girders were encased in gunite applied by the Fritz-Rumer-Cooke Company, all without in any way interfering with traffic, either on the railways or on the street. After the encasement was 28 days old the girders were

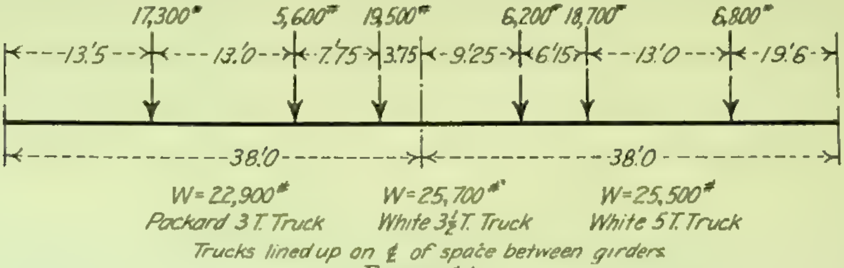


FIGURE 14

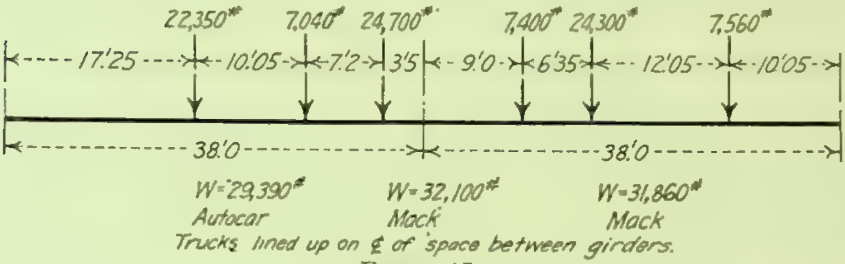


FIGURE 15

again loaded with motor trucks as shown in Figure 15 and deformation and deflection readings taken. The results of these tests are given in Table II following, which shows both the calculated and measured stresses and deflections:

TABLE II
UNIT STRESSES UNDER TEST LOAD

	Load	Calculated			Measured				
		Old DL	New DL	LL	Gird. No.	New DL	% of Cal.	LL	% of Cal.
Before Reinforcing	73200	8740	0	4570	14 15	0 0	0 0	957 1000	20.9 21.9
After Reinforcing	93700	8740	3935	5525	14 15	3161 2044	82.5 53.4	834 1212	15.1 21.9
DEFLECTIONS									
Before Reinforcing	73200			0.34	14 15	0 0	0 0	0.11 0.11	32.4 32.4
After Reinforcing	93700			0.42	14 15			0.10 0.14	23.8 33.3

In making the calculations, the stiffening effect of the floor was neglected which accounts for the comparatively low stresses developed under the test loads. The old floor assists in two ways in carrying the load: it has some strengthening effect on the girders on account of the bond around their top flanges, and it distributes a part of the live load to the other girders of the floor. A comparison of the results of the tests before and after the encasement indicates that the reinforcement has stiffened the girders enough to reduce the deflections under live load about 11.6 per cent and the live load unit stresses about 13.5 per cent.

In addition to being restored to safe section and protected from further corrosion, these girders have been prepared for usefulness with the heavier traffic of the future by the design of the reinforcement for T-20, the heaviest loading specified by the Ohio Division

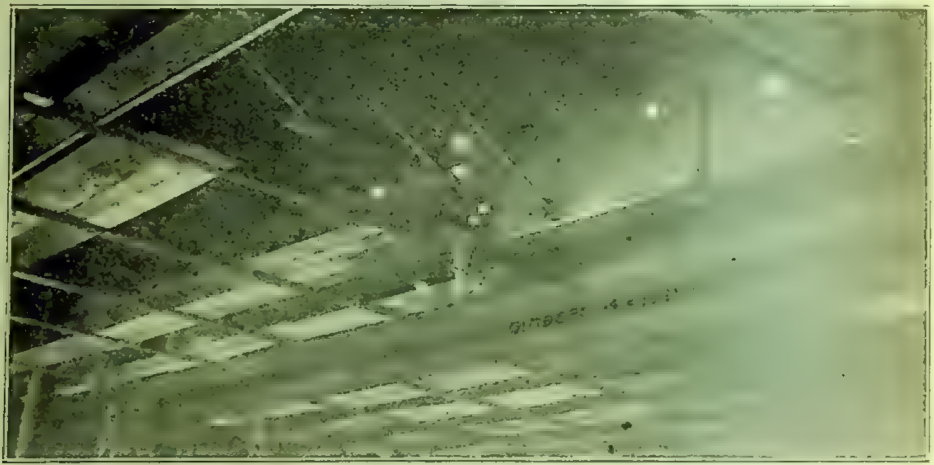


Figure 16

of Highways. The calculated stresses in the bottom flanges with this loading are 8740 lb. per square inch for dead load of corroded steel, 3835 lb. per square inch for dead load of reinforcement, and 5300 lb. live load, making a total of 17,875 lb. per square inch.

Figure 16 shows the appearance of the two test girders after encasement.

Encasement of Steel Columns

In current practice the weight of concrete fireproofing for columns is added to the dead load and the steel made strong enough to carry the entire load. If the strengthening effect of the encasement could be considered in design the saving would be important.

Concrete-encased columns were tested at the National Physical Laboratory in Teddington, England at the time the beam tests previously referred to (page 5) were made. Four columns were about 16 ft. long and four were 20 ft.; the encased section was approximately 8 in. square. The following is quoted from the report which is entitled "Strength of Compound Columns of Steel Joist Section Encased in Concrete:"

"No special precautions were taken in the manufacture of these steel and concrete columns, and the concrete was not rammed into the moulds, it was merely worked around the joists with a shovel, the idea being to reproduce as nearly as possible the conditions frequently existing in ordinary building contract work. . . .

"The average strength of the first two naked steel columns 4 in. x 3 in. in section was 4.59 tons, while the average strength of the first pair of compound columns was 51.6 tons.

"From the measurements of actual compression under the loads I find that out of the total load of 51.6 tons, the 4 in. x 3 in. steel joist section was actually carrying about half of that load, or 25.8 tons, the remaining half, also 25.8 tons, being carried by the concrete.

"It will thus be seen that by the mere covering of the steel joist with the casing of concrete the steel itself was carrying 5.6 times the load which had buckled the naked joist section, and the efficiency of the steel as a load carrier was thus enormously increased.

"In the case of the 5 in. x 4½ in. joists a similar result was obtained, although not to the same extent owing to the covering of concrete being less in relation to the steel section, but the increase in the strength is still very important.

"The average ultimate load which buckled the compound columns with the 5 in. x 4½ in. joists amounted to 43.27 tons, and I find from the measurements of compression that approximately 32 tons out of that load was being carried by the steel leaving about 11 tons to be carried by the concrete.

"In this instance the carrying power of the steel was increased to 2½ times the strength which had been obtained in the tests of the naked column."

An investigation of the strengthening effect of encasement of columns was made at the Engineering Experiment Station of The Ohio State University in September, 1926. The columns tested were 6-in. Bethlehem H sections, 20 lb. per linear foot, 18 ft. 10 in. long, slenderness ratio 150.7, ends milled as is usual in structural shops. Seven pieces were used. Three, 2a, 2b, and 2c, were encased with gunite to a square section 10 x 10 in.; three, 3a, 3b, and 3c, were encased with poured concrete to the same dimensions; one, No. 1a, were tested bare. After being tested, encased column No. 2c was stripped of its gunite covering and tested naked; in this test it is referred to as No. 1b.

All the tests showed that not only did the encasement stiffen the sections greatly, but it also added to the strength.

The columns were tested with square ends between hardened

steel plates. Encasement was omitted for about an inch from the ends of the steel so that the loading was applied to the steel only. During the test, strain gage readings were taken at the four toes of the steel column at the mid-height, some of the concrete being cut away so that the strain gage could be applied directly to the steel. Deflection readings were also taken about both axes. All the encased columns failed by the steel at the ends upsetting when the load had passed the elastic limit.

The following table shows the major results of the tests:

TABLE III
RESULTS OF COLUMN TESTS

Column	Treatment	Max. Ult. Load, lbs.	Max. S.G. Unit Stress on Steel	P/A (Steel)	Load at 16,000 lbs. per sq. in. S. G.	Es/Ec at 16,000 lbs. per sq. in. S. G.
1a	Bare	180,000	30,200	31,000	96,000	
2a	Gunite	221,600	20,500	38,150	166,000	20.6
2b	Gunite	206,750	20,100	35,600	165,000	21.0
2c	Gunite	246,850	22,500	42,350	168,000	20.1
3a	Concrete	191,800	18,300	33,000	167,000	20.4
3b	Concrete	248,000	17,200	42,700	236,000	10.5
3c	Concrete	241,400	18,600	41,500	210,000	12.9
1b	Bare	160,000	30,400	27,500	90,000	

The column "Max. S. G. Unit Stress on Steel" represents the average of the maximum strain gage results expressed as unit stresses based on the modulus of elasticity of 30,000,000. The next column "P/A Steel" represents the unit stresses near the top and bottom where the columns failed, the maximum load divided by the cross-sectional area of the H-column (5.81 sq. in. according to the structural handbook.) The column "Load at 16,000 lbs. per sq. in S. G." represents the testing machine's load on the specimen when the strain gage readings indicate a unit stress of 16,000 lb. per square inch on the steel column. The ratios of the moduli of elasticity give an indication of the relative resistance offered by the encasements.

In none of the tests was there any evidence of the bond's failing except at the ends where the steel was driven down into the concrete after the maximum load was reached.

The two graphs, Figures 17 and 18, show the average of three tests for each type of encasement with theoretical curves for the bare and composite columns computed on the bases of two ratios for moduli of elasticity. It will be noticed that the poured concrete offered more aid to the steel than did the gunite. This is particularly true at low loads. The inclination of the curve for gunite,

Figure 17, is little more than that for the bare beam. It appears that the shrinkage of curing caused an initial stress in the specimen, tension in the gunite and the same amount of compression in

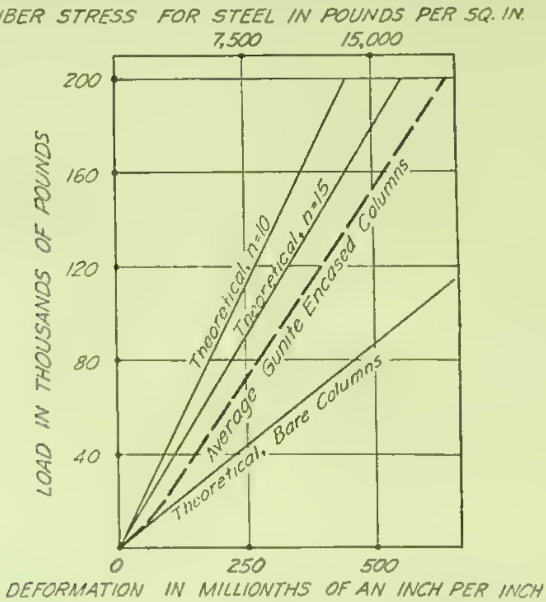


FIGURE 17

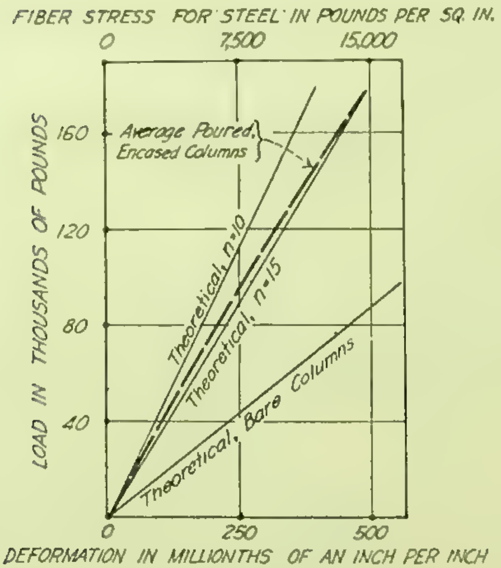


FIGURE 18

the steel. Some load was necessary to offset this tension in the gunite and bring it into compression so as to aid the steel.

The tests reported herein show the assistance offered to a steel column by encasement. This aid is in two forms: reduction of the slenderness ratio and increase of the cross-section. In either case the designer would be much interested in the value to assign to the ratio of the moduli of elasticity.

It is recommended that only the stiffening action be considered in the design of columns until more definite data are at hand to show how much of the load is taken by the steel after a year or two when the flow or readjustment of the encasement has taken place. In all probability much of the load will still be taken by encasement, but more investigation will have to be made before a safe value can be assigned.

DESCRIPTION OF INDIVIDUAL TESTS

CORRODED BUILT-UP BEAMS

Beams XI, XII, XIII, and XIV were 21-in. wrought-iron plate girders from the Adams Street Viaduct in Chicago. Each was composed of a 21-in. plate 3/8 in. thick, and four 3½ x 3 x 3/8-in. angles, and when new had a moment of inertia of 1081.07 in.^{4}; section modulus in compression 107.57 in.^{3} section modulus in tension 98.73 in.^{3} The condition of these stringers after 40 years of service is indicated by the diagrams, Figures 4, 19, 20 and 21, and the notes in Table IV, page 65, showing that Beam XII was probably in best condition and Beam XIII in worst condition. The test span for all beams was 12 ft.

Beam XII, tested bare. Strain gage readings were taken at both edges and middle of each flange. The top of the beam started early to deflect toward the south, and above 75,000 lb. load, side deflection stress exceeded the vertical deflection stress. Figure 22 shows this bowing action clearly. In the bottom the average of the side deflections ran from 1½ to 2 strain gage divisions* lower than the center, with higher readings on the north side indicating a deflection toward the north, and, consequently, a twisting of the stringer. Failure occurred at a load of 84,300 lb. by buckling of the web, the top going toward the south. This buckling was more marked at the east end.

Load deformation curves are shown in Figure 23. Average top and average bottom readings were plotted separately, along with

*A Brown & Sharpe inside micrometer caliper was used in calibrating the strain gage, and the value of one strain gage division was found to be .000180 per 20 in. or .00000945 in. per in. deformation.

theoretical curves for both new and corroded stringer. It is interesting to notice that performance at working loads is higher than theoretical for the corroded stringer, but of course falls below the curve for a new stringer of the same section. Note that at about 10,000 to 12,000 lb. per square inch observed curves begin to fall

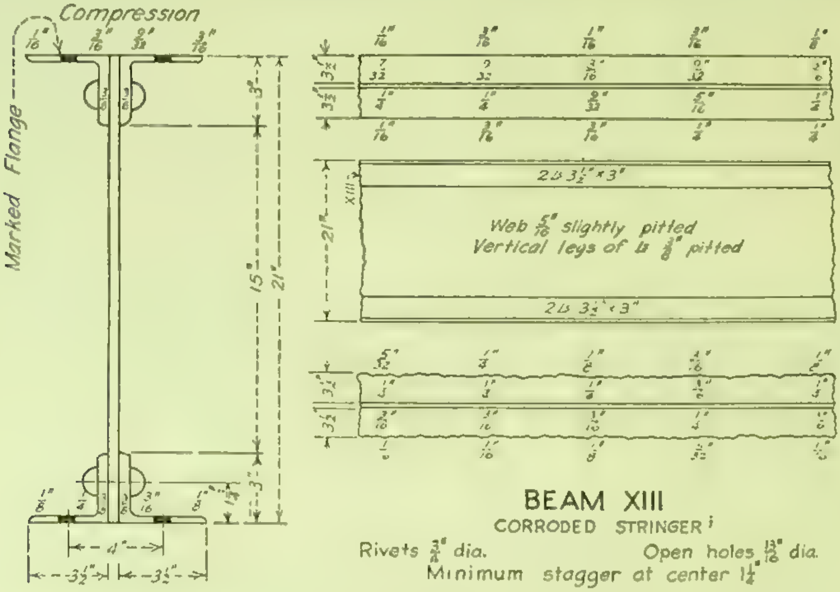


FIGURE 19

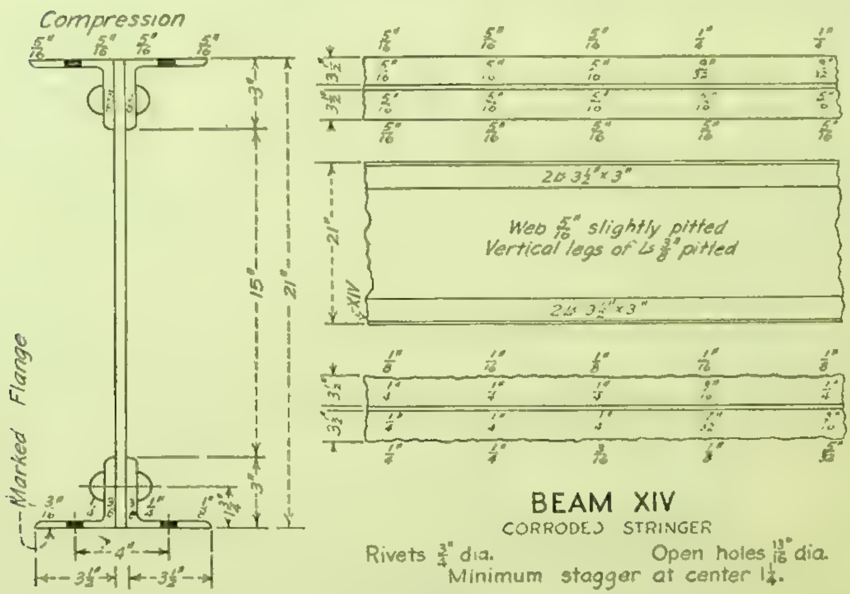


FIGURE 20

away faster from the theoretical curves for the original stringer. This may be due to the cumulative effect of the elastic limit's being exceeded at many small places.

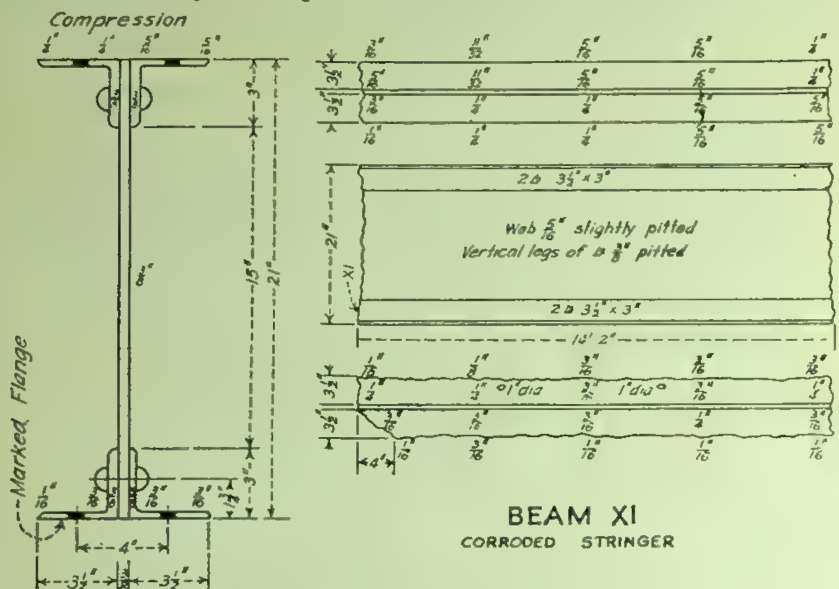


FIGURE 21

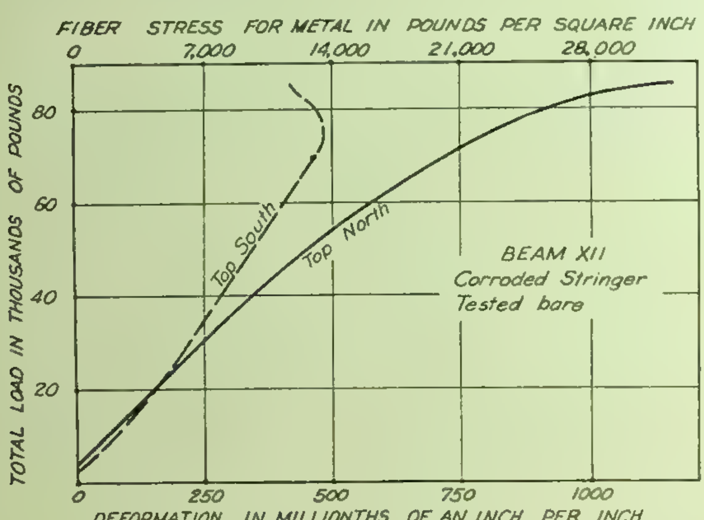


FIGURE 22

These curves are valuable as a basis for judging the behavior of the reinforced stringers. At 14,000 lb. per square inch fiber stress the theoretical loading is 92 per cent of the observed load, at

21,000 lb. the theoretical load is 93 per cent. Corresponding values for the tension side are 98 per cent and 100 per cent respectively. The percentages of excess loads of encased stringers over the loads of bare stringers computed from the caliper measurements may be reduced by these percentages.*

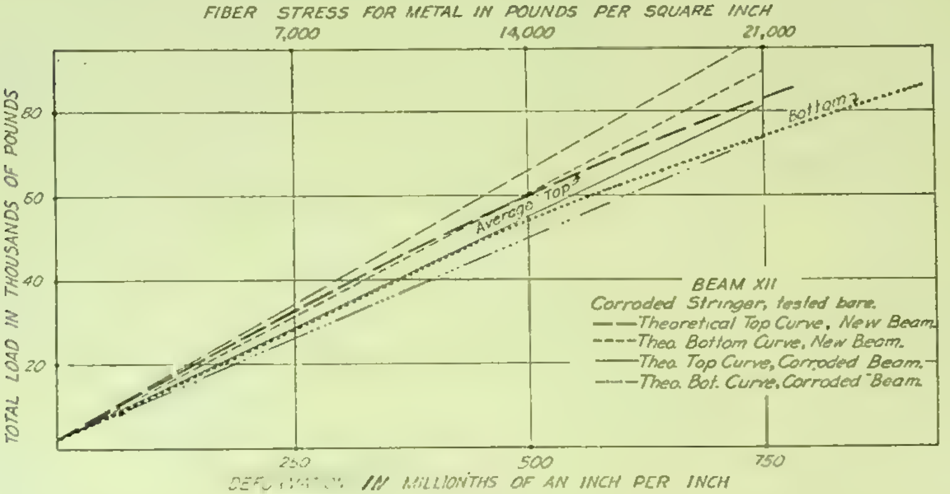


FIGURE 23

Stringer XIII, encased with two inches of gunite. No steel was used except mesh and four 3/8-in. rods to hold the mesh in place. The arrangement was very similar to that shown in Figure 6. Properties of the encased beam, figured on a basis of "n" as 10, neutral axis of composite section computed with gunite in compression only, and all steel, including bars and mesh, working, are:

	Moment of Inertia	Section Modulus Compression	Section Modulus Tension	Neutral Axis
Corroded beam.........	770.76	77.39	69.82	
Encased beam..........	965.8	118.9	75.0	8.12 in. from top

It will be noticed that the compression section modulus for the composite beam is much greater than that of the original because of calculation of the concrete's acting in compression. The tension section modulus for the composite beam is somewhat larger than

* Kennerley Bryan, Jr., engineer of the Fritz-Rumer-Cooke Company, has found that the observed load for 18,000 lb. fiber stress of Beam XII exceeds the calculated load by 5.7 per cent. With this correction applied, results of tests of the other stringers are:

Beam	Load to Produce Fiber Stress of 18,000 Lb.				Increase Due to Reinforcement	
	Calculated for New Beam	Calculated for Old Beam	Corrected (plus 5.7%)	Observed	Over Old	Over New
XI	80,000	61,250	64,750	100,000	54%	25 %
XIII	80,000	57,500	60,750	77,500	28%	— 3 %
XIV	80,000	66,750	70,550	110,000	56%	37½%

for the corroded beam, but does not come up nearly to that of the original beam; this small increase is due to the effect of the mesh and of raising the neutral axis.

Strain gage measurements were made on the top and bottom edges of the web plate. Deflection measurements were also taken. This beam was made March 26 and broken April 2. Seven days is a short time for the concrete to have attained sufficient elasticity for the value of "n" to be taken as 10. It was thought best, however, for all computations to be made on this basis.

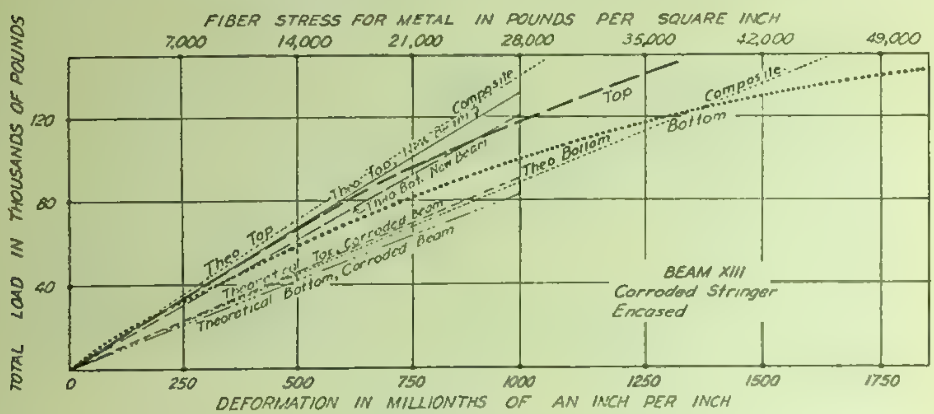


FIGURE 24

The first crack in the gunite was noticed at a load of 31,200 lb., cracks appeared across the bottom at 56,200 lb., and the final break came at 164,300 lb. The scale beam dropped in about 30 seconds at a load of 61,200 lb., and very rapidly above 131,200 lb. The beam deflected vertically, and the failure was indicated by additional loading's not keeping the scale or weighing beam up. After failure, cracks showed in a great many places, few spaces between cracks at the bottom flange between load points being greater than 3 in. Cracks were nearly vertical over the bottom flange, and sloped on the web toward the midpoint, greater slope near the supports indicating the effect of shear. One large crack showed on the south side where the final failure occurred. There were no horizontal cracks under the top flange.

Figure 24 shows the load deformation curves for stringer No. XIII with the theoretical curves for the corroded beam, the original new beam, and the composite beam. Both test curves are above the theoretical curves for the bare corroded beam, that is, both test curves show greater loads carried at a given unit stress. This indicates the aid given by the covering. The theoretical curves for the composite beam are shown wide apart; the curve for the

top is above the test curve except for a few thousand pounds at low loads, and the one for the bottom is well below all along. This is the way curves for reinforced concrete beams usually look. To consider no gunite or concrete acting in tension, particularly for the tension side, seems to be an error on the side of safety, for even up to rather high loads it appears that a considerable amount of the gunite acts in tension. If some tension were allowed for the gunite the neutral axis would be lowered, the moment of inertia somewhat increased, the compression section modulus lowered, the tension section modulus raised, and the two theoretical curves brought somewhat closer together. The test curve for the top

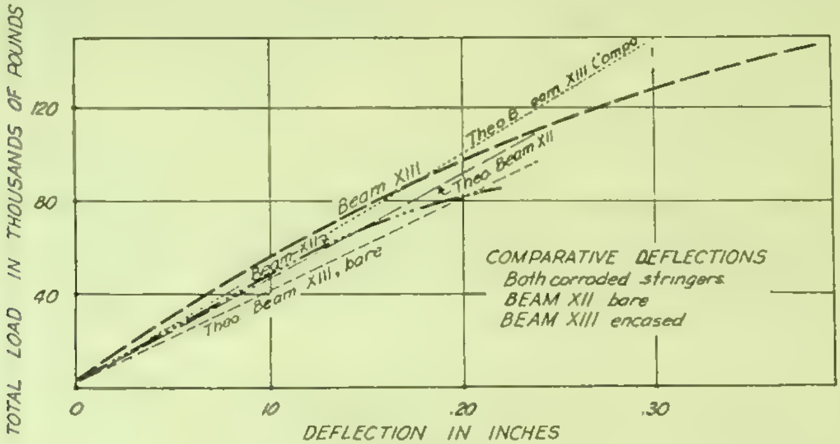


FIGURE 25

would then be thrown in a more favorable light, and that for the bottom would not fall below the theoretical prediction.

Comparative deflections for Stringers XII and XIII are shown in Figure 25. The observed curve for bare Beam XII runs very well with the theoretical up to ordinary working loads, but does not exceed the thoretical to as large a degree as did the fiber stresses. The observed curve for Stringer XIII appears to bear about the same relation to its theoretical curve, except that it runs slightly stronger for the smaller loads. Both curves for Beam XIII run well above those for the bare stringer, showing the marked stiffening effect of encasement.

Stringers XIV and XI were reinforced corroded plate girders, both 28 days old at the time of testing. Reinforcing steel rods designed to give the member a value equal to that of a new beam of the same dimensions were placed near the bottom flange of each stringer. The bars of Beam XIV are shown in Figure 26. Figure

7 shows the arrangement in Beam XI with both pairs of bars bent up at 45 degrees at small pivots and welded to the top flange at the ends.

The properties of these beams were:

Beam		Moment of Inertia	Section Modulus Compression	Section Modulus Tension
XI	corroded	804.43	84.23	70.26
	reinforced	1302.7	154.8	103.4
XIV	corroded	845.22	87.77	74.34
	reinforced	1385.5	161.1	111.7

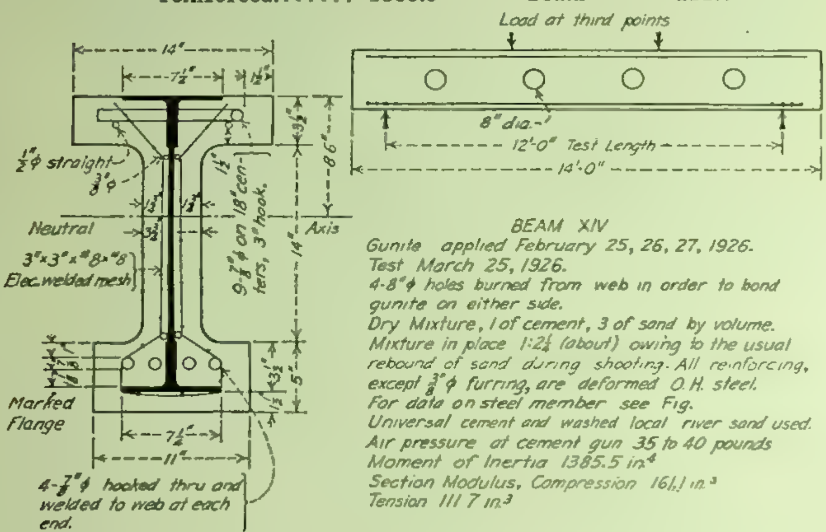


FIGURE 26

For testing these beams strain gage points were prepared on the top edge of the web of the stringer, about ½ in. in from the edge of each top flange angle, about 1 in. in from each edge of the top flange of gunite, and on the edge of the web plate at the bottom.

During the test of Beam XIV the scale beam dropped slowly at a load of 107,600 lb. and in 2 or 3 seconds at a load of 137,600 lb. At 192,600 lb. the last set of readings was taken as the top was bowing decidedly. The final failure came at 225,700 lb. when the total deflection was ½ in. At the end of the test, cracks were noticed about every 5 in. all along the vertical face of the bottom flange, particularly on the south side. A number of them ran up into the web, becoming more inclined toward the center as they approached the reactions. The final failure caused a longitudinal break at the north face of the top flange near the center for a distance of three or four feet. At about 120,000 lb. load it appeared that the concrete had broken loose from the web at the west end.

Beam XIV started bowing at the beginning of the test. At 14,000 lb. per square inch the gunite on the north edge of the top deformed about 20 per cent more than the average and that on the south 20 per cent less than the average. This condition would tend to cause higher unit stresses for the same loads and failure at a lower load than would obtain had the beam remained straight. Possibly it was due to the difficult set-up. Supporting girder, test beam, and top girder made a total height of more than ten feet in the machine.

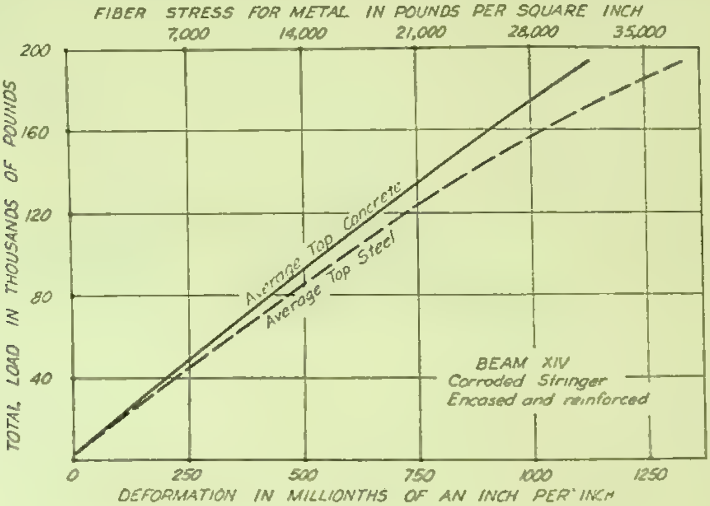


FIGURE 27

As shown in Figure 27 the steel at the top showed greater deformation than the concrete for the same load. Loads were applied directly to the steel and not to the gunite which was obliged to get its deformation through bond to the steel, and which the curves show to have been lagging a bit. At 21,000 lb. per square inch stress the lag is about 67 millionths of an inch per inch, or about 1/100 in. for the entire 12-ft. span. It appears that this lag was in the gunite itself and not due to bond failure because there was no evidence of the gunite's being loose from the steel. This indication of lag may apply only to the 20 in. covered by the strain gage and not to the entire span.

Figure 28 shows the average of all top and bottom strain-gage readings as compared with theoretical curves for the new stringer, composite beam, and corroded bare stringer. Both test curves are well above theoretical curves for corroded bare beam and original new beam. The test curve for the top is below the theoretical for

the top of the composite beam, while that of the bottom is well above all through the working part of the test, conclusive evidence that the computed neutral axis was too high. A mean of computed and observed curves would show the test curves slightly stronger up to ordinary working loads. The test curves show the neutral axis below the mid point up to a stress of about 10,000 lb. per

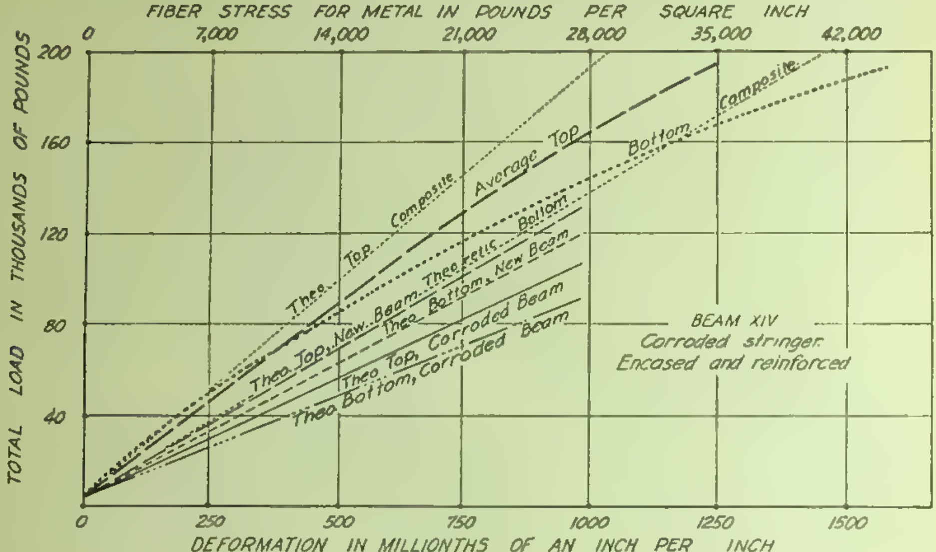


FIGURE 28

square inch. Moreover these comparisons are with the curves of the theoretical composite beam which are above the original beam values and may be considered rather severe on the test beams, as the attempt was to reinforce the corroded beams to meet the original beam values and not to exceed them.

Beam XI tested considerably better than Beam XIV. The beam deflected vertically without any tendency for the top to buckle sidewise. The first crack occurred at a load of 64,950 lb. The scale beam was first noticed to settle at 74,950; it fell in about 30 seconds. At 118,950 it dropped in 10 seconds, and continued to require 10 seconds up to 168,950 lb., but began to drop more rapidly at heavier loads. The last readings were taken at 193,950 lb., and failure occurred at 227,950. There were a number of cracks in the north vertical face of the bottom flange, many of which ran up into the web. At failure a break very similar to that of Beam XIV occurred in the gunite of the top flange. Figure 29 shows that the deforma-

tions of top steel and top gunite agreed closely, indicating that the bond held.*

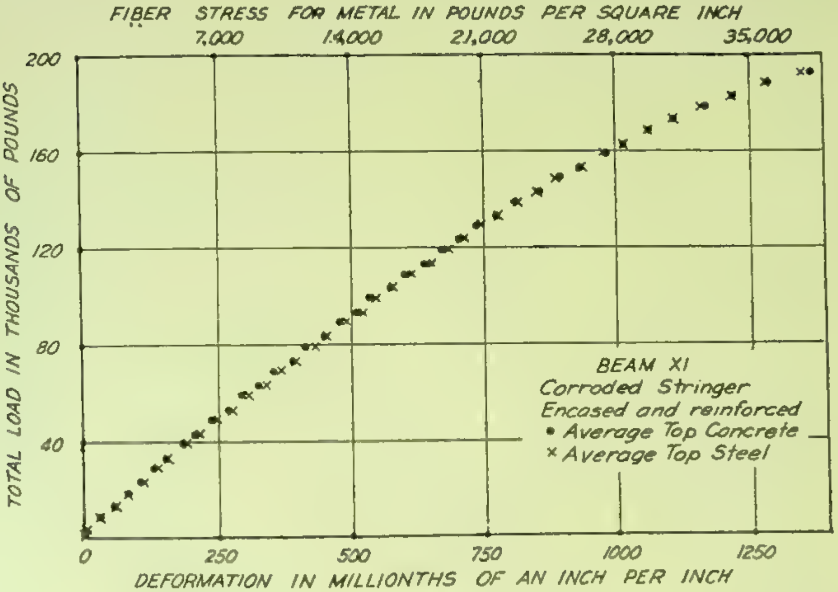


FIGURE 29

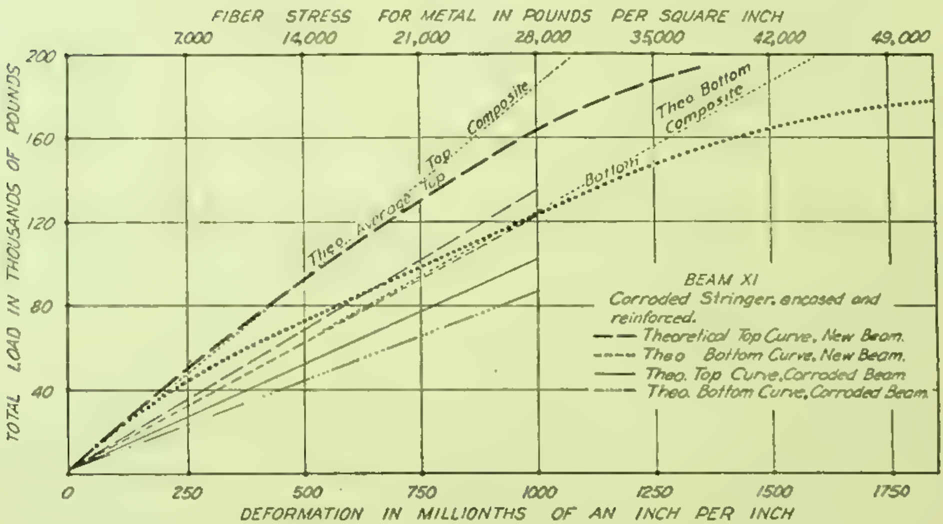


FIGURE 30

As the test procedure of Beam XI was much more regular than that for No. XIV, it might be expected that the test of the former

*The use of points instead of lines in Figure 29 gives an idea of the number of observations taken in a test and their agreement with each other. The curves were omitted because they would have been so close together.

would show more accurate and dependable results than that of the latter.

Figure 30 shows average fiber stresses for Beam XI compared with the theoretical curves for the original new beam, for the composite beam, and for the corroded bare beam. When compared with those for Beam XIV these top curves look much the same, but the bottom curves are quite different, showing much greater stress in XI for the same loads. It appears that the placing of the rods all low in the case of Beam XIV helped to keep down the stresses for the bottom flanges. The top test curve of Beam XI runs over the

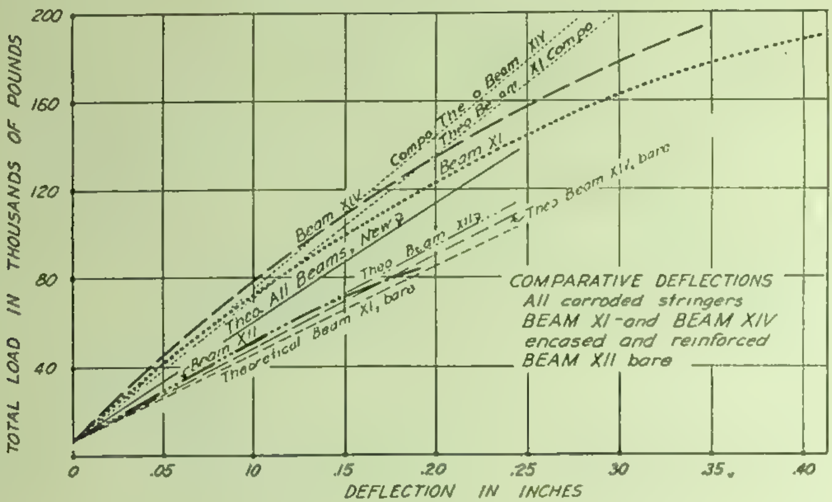


FIGURE 31

theoretical for the composite up to low working loads, and shows the neutral axis to have been at no time below the mid point, more than an inch higher than for XIV. As the failures were all in bending, it is questioned whether the bent-up bars of Beam XI had any appreciable effect on the test.

Comparative deflection curves for Stringers XII, XIV and XI, together with curves for the original new beams and for the computed composite beams, are shown in Figure 31. It will be noticed that bare stringer XII had a larger moment of intertia than bare beams XI or XIV, yet, of course, tested far below the value of the original new beam. Reinforced beams XI and XIV were both stiffer than the original new stringers, and, up to low working loads, were stiffer than the theoretical composite stringers, the theoretical curve of stringer XIV meeting the test at 16,000 lb. per square inch, the theoretical of XI meeting the test at about 14,000 lb.

The close agreement between the computed and observed deflections is assurance that the strength of such composite beams may be safely predicted for working loads. Of course some variations are to be expected as the theoretical curves for the composite beams, figured by the transformed section method, depend on the assumption of correct values for the ratios of the moduli of elasticity.

TESTS OF OLD I-BEAMS

The tests of beams XV, XVI, XVII, and XVIII were largely duplications of those on the stringers. The I-beams were originally 24-in., 80-lb. sections with moment of inertia, 2087.2 in.^{4} and section modulus 173.9.[3] The modulus of elasticity of the steel was assumed to be 30,000,000. The length of span was 13 ft.

Figures 32, 33, 34, and 35 show the corroded condition of these I-beams. Caliper notes of the four beams are given in Table V, page 69.

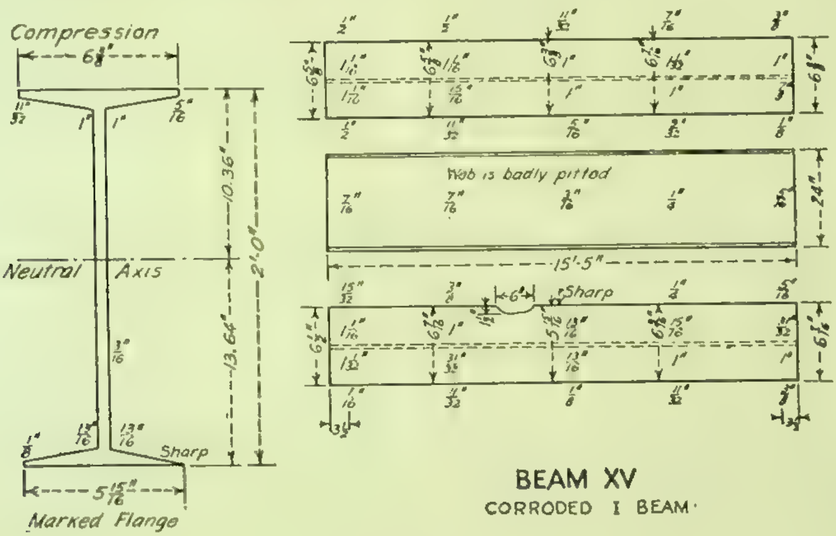


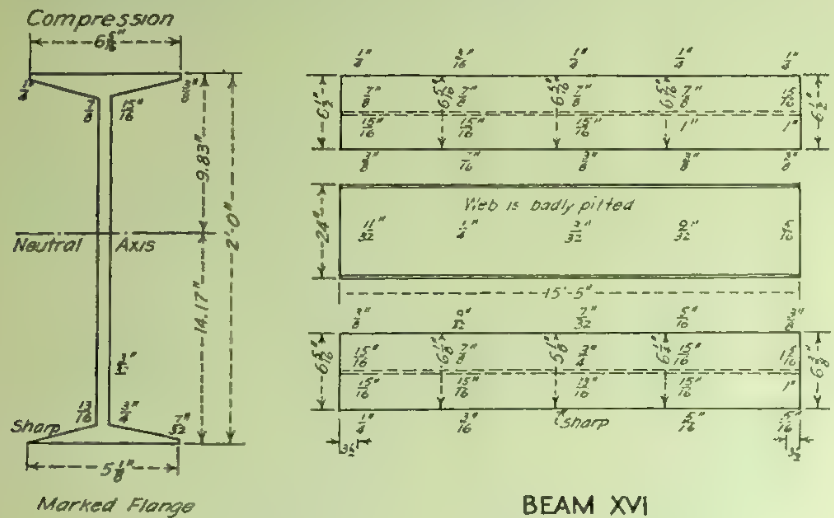
FIGURE 32

Beam XV was tested bare; Beam XVI, encased without reïnforcement, was tested at 7 days; Beam XVII was reinforced and tested at 7 days; Beam XVIII was reinforced and tested at 28 days.

Test of Beam XV. Deflection readings were taken at mid span. At the top extensometer readings were taken near each edge of the flange; at the bottom in the center. The top readings show a slight bowing toward the north. At a load of 115,400 lb. the scale beam began to drop. At 120,400 lb. it began to drop rapidly. The

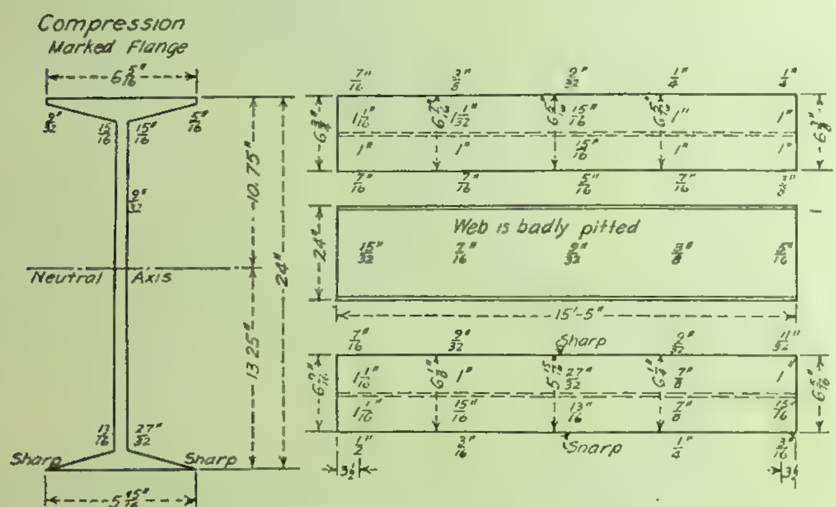
last readings were taken at 140,400 lb. when the deflection was 0.69 in.; at 146,825 lb. the beam failed to take more load.

Figure 36 shows the theoretical and load deformation curves for I-beam XV, the higher observed curves indicating rigorous in-



BEAM XVI
CORRODED I BEAM

FIGURE 33



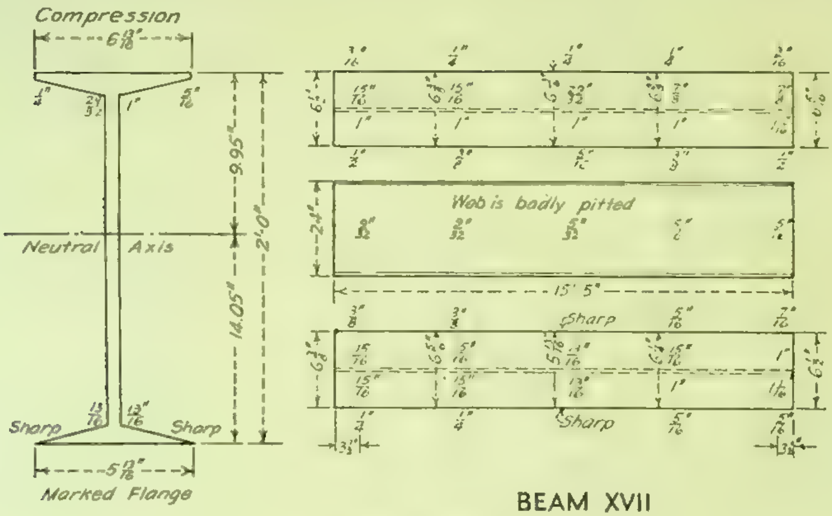
BEAM XVIII
CORRODED I BEAM

FIGURE 34

spection in making the caliper measurements. At a unit stress of 15,000 lb. per square inch the top is 21 per cent more than the theoretical and the bottom is 28 per cent above; at 22,500 lb. per

square inch these values are 16.5 per cent and 24 per cent respectively. Again, as in the case of Beam XII, we have correction figures which may be applied in computing the excess strengths of the encased beams.*

Beam XVI like Stringer XIII was encased without any attempt at reinforcement to give it the strength of the original section. (Figure 6 shows the cross-section of this member.) Strain gage readings on the steel were taken on the top near each edge of the



BEAM XVII
CORRODED I BEAM

FIGURE 35

flange, and in the center at the bottom. During the test the top flange bowed toward the north. Hair cracks appeared at a load of 51,200 lb., web cracks on the north at 96,200 lb., and on the south side at 131,200 lb. The scale beam started to drop at 86,200 lb. At 146,200 lb. a cracking sound was heard. A horizontal shear crack appeared on the north side at 156,200 lb. Readings were discontinued after the loading of 166,200 lb. During the attempt to add a 5,000-lb. increment a cast iron block above the upper steel loading girder broke, consequently no final failure load was re-

* A load of 85,000 lb. was required to develop 18,000 lb. fiber stress in Beam XV, an increase of 21.4 per cent over the calculated load of 70,000 lb. Kennerley Bryan, Jr. (see note page 25) estimates from this test the following values for the remaining stringers:

Beam	Load for 18,000 lb. Unit Stress				Increase by Reinforcement	
	Calculated Bare	+21.4%	Observed	New	Corroded	New
XVI	55,000	66,750	92,500		38%	
XVII	65,000	79,000	152,500	127,500	93%	19½%
XVIII	70,000	85,000	152,500	127,500	78%	19½%

corded. After the load was removed the cracks were observed to be as in the stringer tests: close together along the flange and sloping more and more toward the center as they approached the reactions. The cracks on both sides of the beam had much the same appearance.

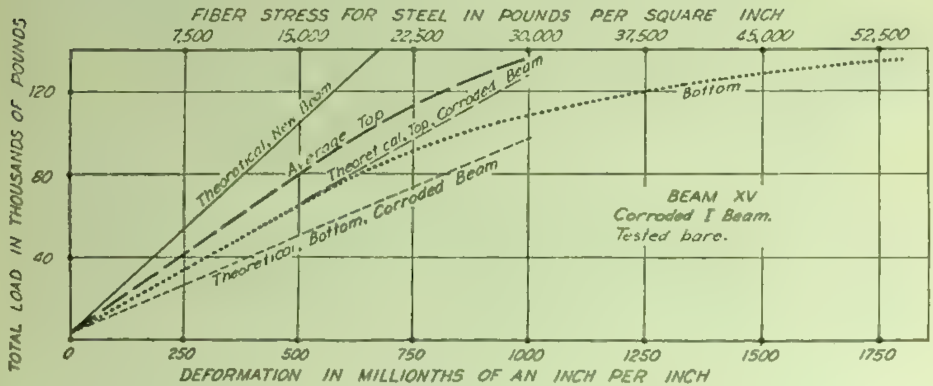


FIGURE 36

Figure 37 shows top and bottom deformations compared with theoretical curves for various conditions of the member. It can be seen that the test had been completely run before the block broke, as the bottom flange began to give way at a load of about 135,000 lb. The high neutral axis used in computation (7.55 in. down from the top,) causes a wide divergence between the two theoretical

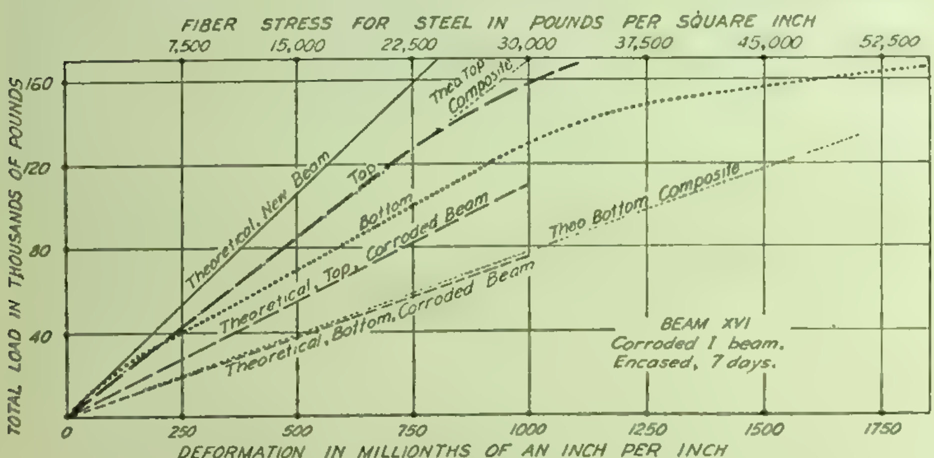


FIGURE 37

curves for the composite beam. The observations show that up to about 6,500 lb. per square inch the actual neutral axis was below the mid point; from this stress up to 28,000 lb. per square inch the

neutral axis was somewhat above the mid point, but not nearly up to the computed value, indicating strongly that much of the gunite was acting in tension. A mean between the two theoretical curves for the composite beam would show both test curves well on the safe side.

Beams XVII and XVIII were reinforced designedly to make their strengths equal to those of the beams as originally rolled. The amount of reinforcement is shown in Figure 38. Beam XVIII

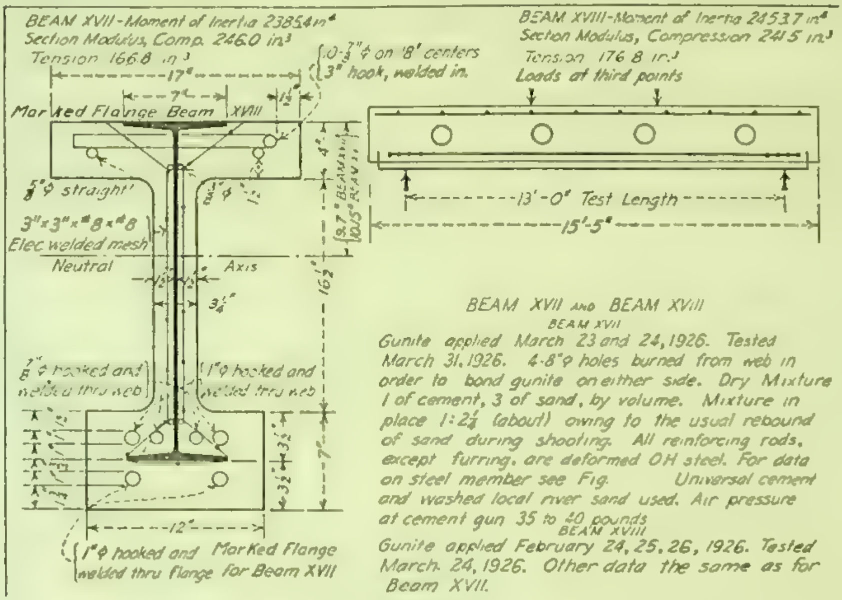


FIGURE 38

was tested when 28 days old, Beam XVII at 7 days; as the two were reinforced in exactly the same manner a comparison of results should indicate the efficacy of 7-day curing.

For Beam XVIII extensometer readings were taken on top at the center of the I-beam and toward the outer edges of the gunite. At the bottom, readings were taken in the center of the flange and on each of the two 1-in. rods. Figure 1 shows this beam in the machine. The first cracks appeared at a load of 57,750 lb. At 97,750 lb. the gunite seemed to loosen from the steel at the extreme west end. At 142,750 lb. the scale beam began to drop; it held for only about 3 seconds at 162,750. When a load of 237,550 lb. had been reached it was noticed that the top steel loading girder had tipped sidewise somewhat and was pressing against the cast-iron guide columns of the testing machine; the load was released, the condition adjusted, and the load applied again in an endeavor to break

the test beam. At 252,400 lb. the east loading plate crushed into the beam pressing one side of the I-beam flange down and breaking out the gunite to the side.

After the test a wide crack was found across the bottom flange at the east strain gage point. Many smaller cracks ran up across the sides of the bottom flange. A crack in the form of an arc about 2 ft. long with center 6 in. to a foot above the top of the beam was noted on the south side near the east strain gage point. Near the west end of the north side of the web were several inclined cracks.

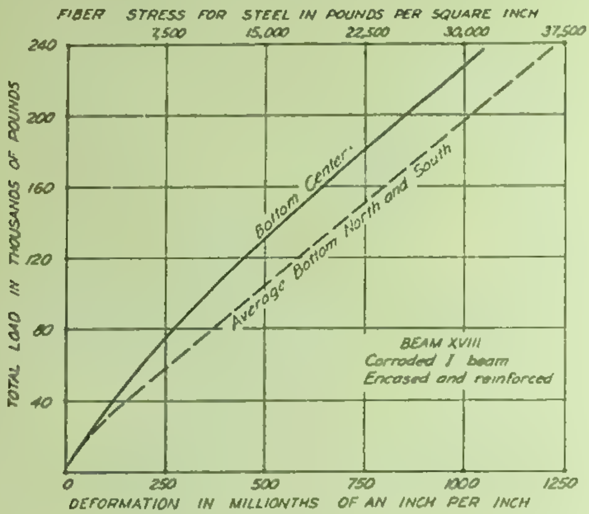


FIGURE 39

Figure 39 gives a comparison of the deformations of the lower steel rods with those of the bottom of the I-beam. The distance from the computed neutral axis to the bottom of the I-beam is 89 per cent of that to the outside of the bars. The test showed that this ratio was 77 per cent at 120,000 loading and 81 per cent at 160,000, so if the straight line theory is considered the tests indicate a neutral axis 8.6 in. lower than as computed at 120,000 lb., and 6.4 in. lower at 160,000 lb., certainly rather low. Two and 7/8 in. lowering would bring it to the mid point between the extreme top and the center of the rods. This all goes to show that the bond between steel and gunite held. The lower curve, of course, more nearly represents the outer fiber stress.

Figure 40, load-deformations for Beam XVIII, shows curves for the top (average of steel and gunite readings at the same elevation) and bottom center (on I-beam only) to be very much in agreement, indicating that the neutral axis was near the center of the beam.

Readings on gunite and steel at the top were close together throughout the test. The "Average Bottom North and South" of Figure 39 is not repeated because it so nearly covers the theoretical bottom composite. The coincidence of the theoretical curve for the bottom of the composite beam with the curve for the new beam shows the closeness with which the design was made to give the old beam the same strength as that of a new beam. The top test curve's relation to the theoretical composite curve resembles that

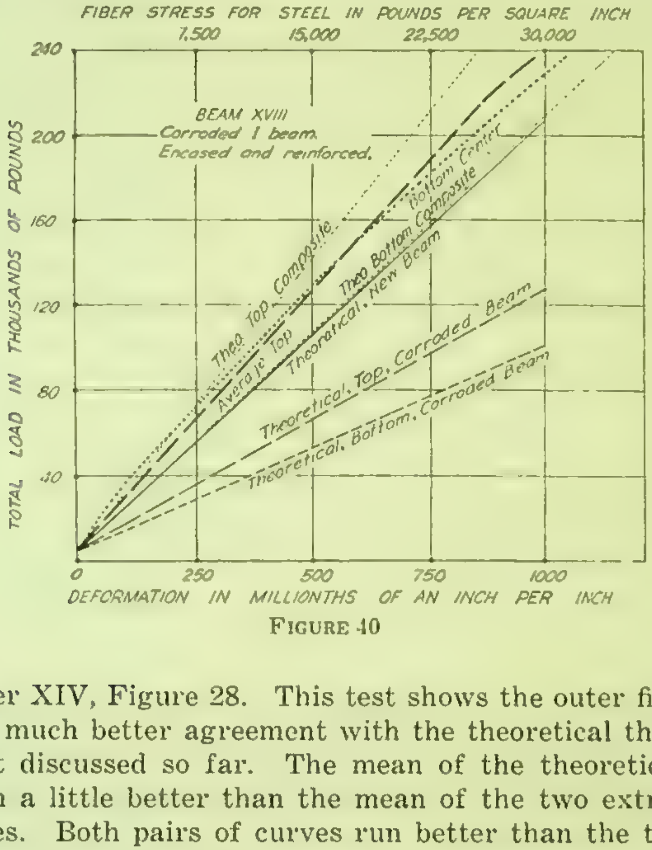


FIGURE 40

of Stringer XIV, Figure 28. This test shows the outer fiber stress curves in much better agreement with the theoretical than in any other test discussed so far. The mean of the theoretical curves would run a little better than the mean of the two extreme fiber test curves. Both pairs of curves run better than the theoretical for the original new beam.

In the test of Beam XVII, strain gage readings were taken at five different places on the top—on the steel at the center and at the edges of the flange, on the gunite at each outer edge. Bottom readings were taken on the I-beam at the center and on each of the two lower rods. During the test the top flange deflected decidedly toward the north. The scale beam was first noticed to drop at 47,700 lb.; the first crack appeared at 62,700 lb.; cracks crossed the bottom flange at 112,700 lb.; at 122,700 lb. the scale beam dropped in 45

seconds; web cracks appeared at 167,700 lb. The scale beam dropped in 15 seconds at 252,700 lb. and in 8 seconds at 282,700 pounds. Readings were taken up to 297,700 lb. The highest load obtained was 328,300 lb. Many small cracks were on the sides of the bottom flange, none greatly enlarged; there were few web cracks. The final failure came suddenly, the gunite of the center of the top flange breaking for a distance equal to half the length of the beam. the whole beam buckling toward the north. This final load was de-

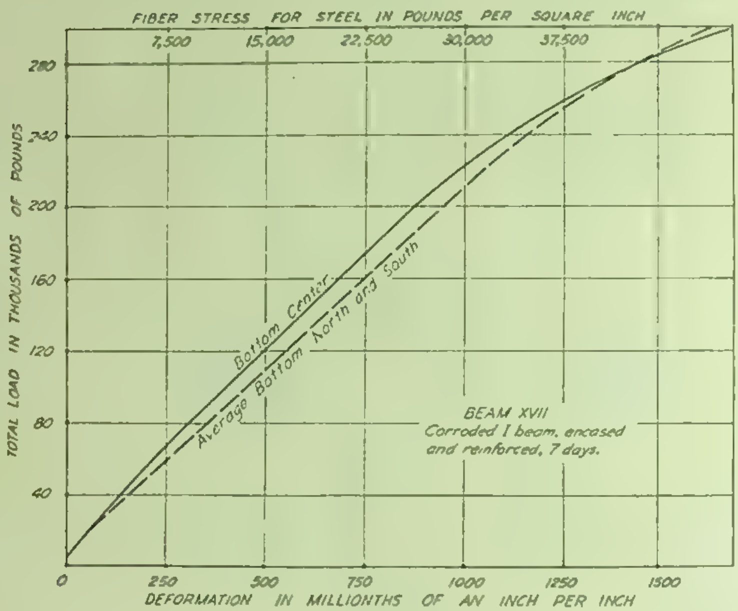


FIGURE 41

finitely the ultimate for the composite beam, and failure was definitely the destruction of the gunite T-head.

Figure 41 shows a comparison between the readings taken on the tension flange of the I-beam and the average of the readings taken on the reinforcing rods. It will be noticed that the readings on the bottom center (on the I-beam) are 89 per cent of those on the reinforcement rods at 120,000 lb. load, and 91 per cent at a load of 160,000 lb. These loads were chosen because in the region of 15,000 lb. per square inch and 22,500 lb. per square inch. If these values were used as a basis, the straight line theory would indicate that the positions of the neutral axis were 15.9 in. and 19.5 in. respectively from the reinforcing rods. The composite beam computations give 16.05 in. This coincides rather closely, closer than the similar observation on Beam XVIII. In both cases it is pretty

clearly shown that the bond was in good condition at working loads. Welding the ends of these rods to the I-beam would cause some tension in the rods but if the bond were not working properly the rods would probably show lower values than they do. Figure 41 shows that at high loads some sort of failure did occur. After the stress exceeded 30,000 lb. per square inch the deformation in the rods began to fall off and toward the end was less than that of the I-beam flange. Apparently the rods were straightening out at the

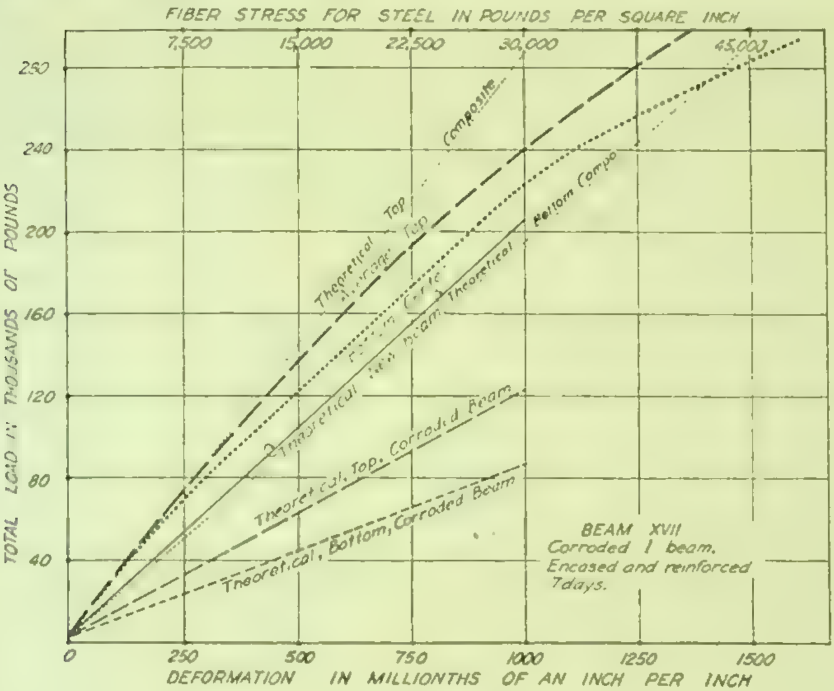


FIGURE 42

bends and were averaging their stress because of a failure of the bond. This phenomenon would have appeared earlier in the test if the bond had given away earlier.

Figure 42 shows the load deformation curves from Beam XVII. The observed curves keep well above the theoretical of the original beam up to the elastic limit. The curve for the top is the average of all the top readings. The curve for the bottom is for the strain gage readings on the I-beam at the center only. Here, as in the case of I-beam XVIII, the lower curve on Figure 41 should be kept in mind as representing the outer tension fiber stress conditions. The curves, particularly the top, are very similar to those on I-beam XVIII. If the means of the test curves and of the theoretical com-

posite curves were considered, it would be noticed that the test curves would keep above the theoretical until well over ordinary working loads. Both means would be well over the theoretical curve for the original new beam.

I-beam XVII was tested at 7 days and XVIII at 28 days. The results bring out rather clearly that there was very little gain in the strength of the gunite after 7 days. No attempt was made to keep any of the specimens wet after 7 days.

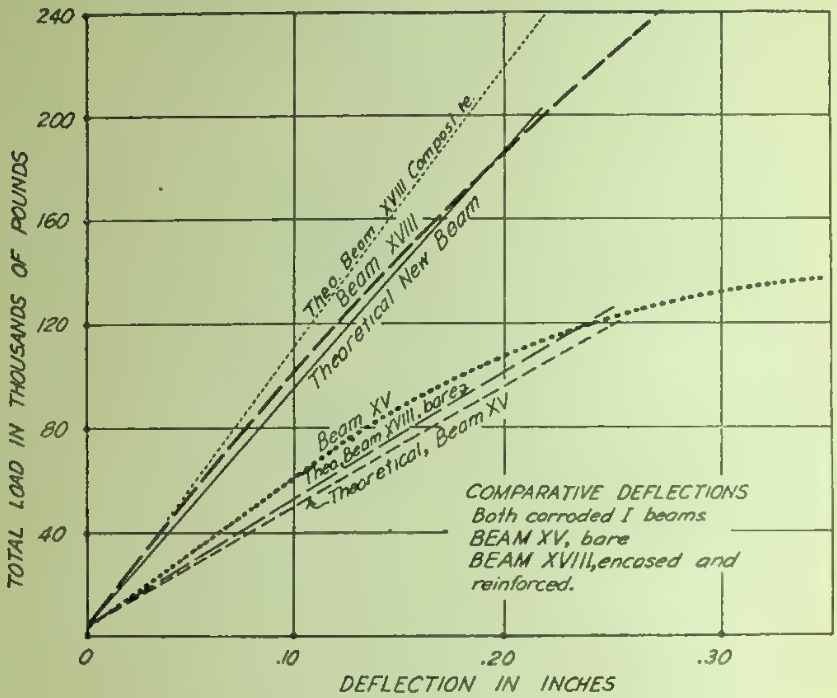


FIGURE 43

DEFLECTION OF I-BEAMS

Figures 43 and 44 show deflection curves for all I-beam tests. The most striking thing is that those of the reinforced encased I-beams XVIII and XVII are so similar in general shape to the composite theoretical curves. I-beam XVIII appears to be slightly stiffer than XVII but so does its theoretical curve. The I-beam curves do not show up quite so well with reference to the composite curves as do those of the girders. It will be noticed, however, that there is more negative section in the girder which would tend to lower the moment of inertia and bring down the theoretical curves. I-beam XVI seems to run very much over its theoretical curve. Excess over the theoretical was noticed also for the fiber stresses. (See Figure

37.) This may be due to the fact that there was aid by the gunite's acting in tension. For the sake of comparison one must observe that naked I-beam XV also tested beyond its theoretical curve.

GENERAL DISCUSSION, STRINGERS AND OLD I-BEAMS

Most of the discussion heretofore in this report has been based on whether the beam as reinforced had attained to the values as computed. In general it may be stated that in every case the application of gunite encasement strengthened the beam, and that

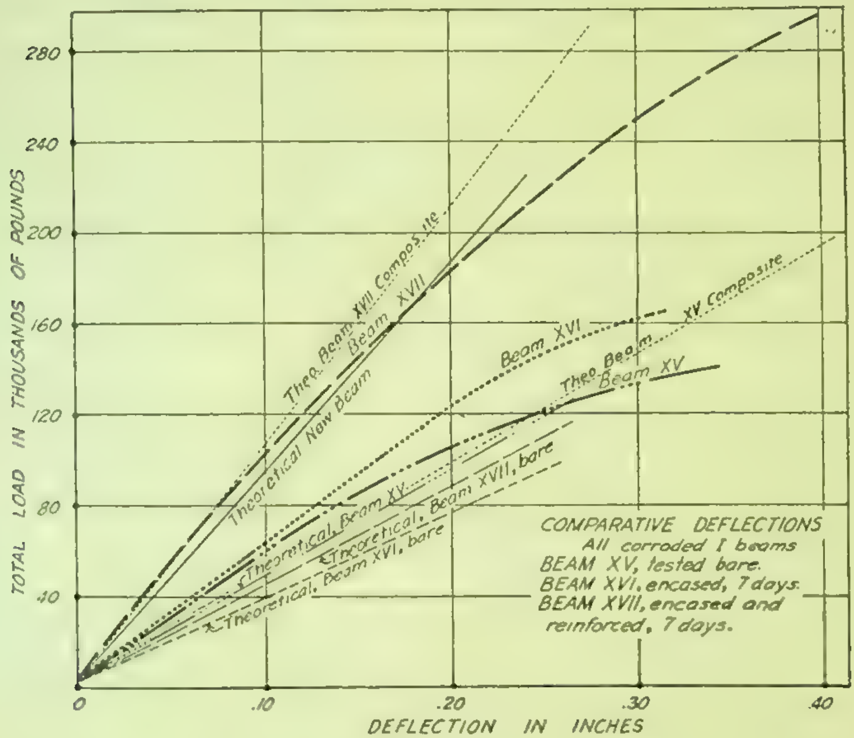


FIGURE 44

in all cases when an attempt was made to reinforce the gunite the result was increase of the strength at working loads over the strength of a beam of full section. There was no indication that the bond ever failed within working loads enough to affect the strength of the beam. The ultimate load for the encased or reinforced beams was larger than that of the unencased beams because of the stiffening of the web and top flanges.

Figures 10 and 11 are summary curves for the stringers and I-beams. The basis in all cases is the straight-line curve for the corroded stringers and I-beams. The ordinates plotted were taken

by means of dividers from the curve sheets already discussed. They were drawn so that a comparison of all the tests could be more easily made. Adjustments due to the difference between the

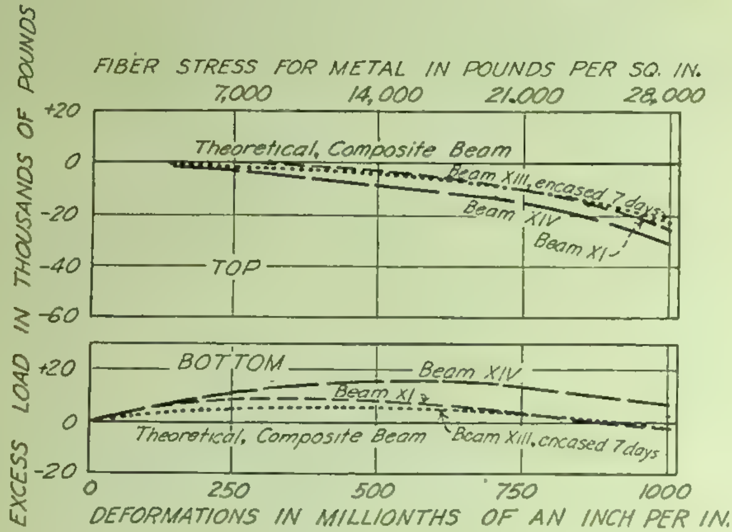


FIGURE 45

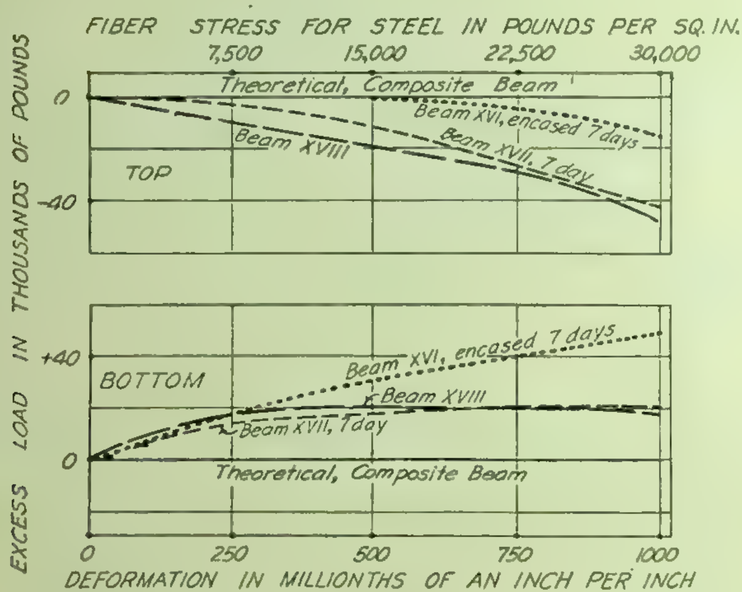


FIGURE 46

bare beams theoretical and as observed can easily be made by eye or by dividers.

Figures 45 and 46 are based on the theoretical straight-line

curves computed for the composite beams. The inaccuracy of the computed location of the neutral axis shows up here clearly. An adjustment might be made on these curves for a new location. A change of inclination of the base line could be made so that the curves for both reinforced stringers and reinforced I-beams would run stronger, for working stresses, than the theoretical. Such a change could be brought about by a change in the location of the neutral axis. A reconsideration of the values of the ratios of the

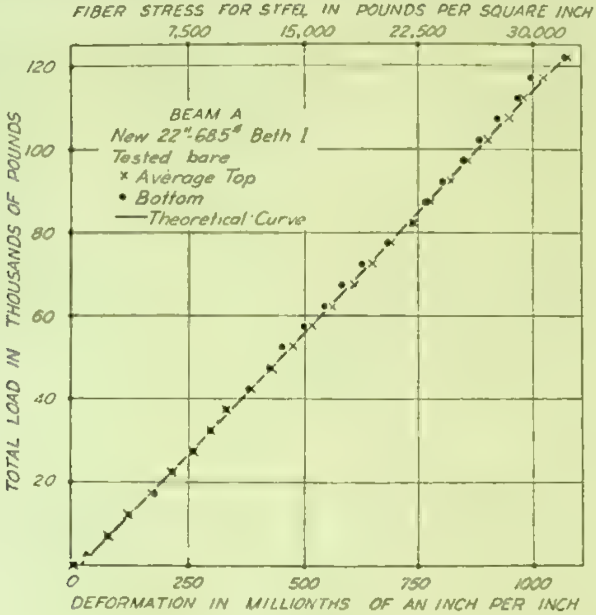


FIGURE 47

moduli of elasticity might make a considerable improvement. More light on this question may be obtained from the column test data.

TESTS OF NEW I-BEAMS

Beams A, B, and C were new 22-in., 68.5-lb. Bethlehem I-beams, moment of inertia 1629.3, section modulus 147.69. Test spans were 19 ft. long. The object of testing these new beams was to obtain data on the strengthening effect of protective encasement such as is generally used against fire and corrosion.

Beam A was tested in a naked condition. Extensometer readings were taken at the edges of the top flange, and at the center of the bottom. Deflections were read at the middle of the span. The top flange bowed toward the north during the test, and the action at failure was a sidewise buckling of the flange. Figure 47

shows fiber stresses and theoretical curves. At a load of 122,000 lb. the scale beam dropped in 5 seconds. Failure came at 140,850 lb.

Beam B was encased in gunite applied over 3 x 3 in. Number 8 by Number 8 electrically-welded mesh held in place by 3/8-in. rods as for Beam XVI, Figure 6. The section was made 12¼ x 5 in. at the flanges, 4 in. through the web, and 27 in. deep over all, that is 2½ in. over top and bottom flanges. The moment of inertia was 2190.5 in.^{4}, compression section modulus 206.3 in.^{3}, tension

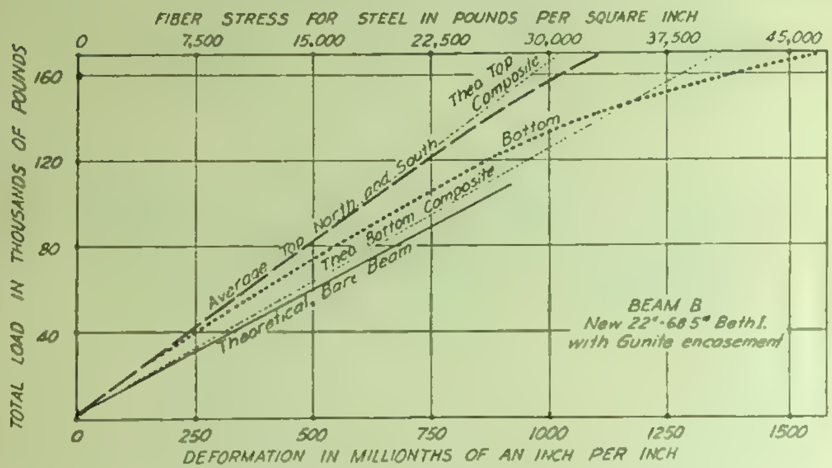


FIGURE 48

section modulus 157.6 in.^{3}, and neutral axis 10.61 in. from extreme top.

In the test of Beam B top readings were taken on the gunite at the edges, bottom readings on the steel. There was some horizontal deflection of the top. The first crack was noticed at 41,400 lb. At 56,400 the scale beam began to drop. It dropped rather sharply at 151,400. Failure came at 222,050 lb., when the total deflection was 1.51 in. Many fine cracks were noticed at the edges of the bottom flange and extending up into the web, but none was very long or large. Some were observed 5 ft. from the west support, that is, beyond the load point and in the region of high shear.

Figure 48 shows load deformation and theoretical curves for Beam B. At 15,000 lb. per square inch the load is 23 per cent over that for the original new beam, at 22,500 lb. per square inch 18.9 per cent excess, observed stresses being computed from bottom deformations. Top deformations were less than bottom, indicating a neutral axis above that of the naked beam. Increase of tension section modulus was 6.7 per cent, of compression section

modulus 39.7 per cent or very nearly the excess load observed at a stress of 15,000 lb. per square inch in the bottom.

It will be noticed that the test curve for the top runs very close to that computed for the composite beam. This cannot be interpreted as stress, only as deformation, since the measurements were made on the concrete and not on the steel. The bottom test curve runs well above that computed, much as was noticed for the other beam tests, showing the tensile action of the gunite.

Beam C, around which mesh like that used for Beam B was wrapped but not held in place with rods, was encased in poured concrete to make a rectangular section 12½ in. wide and 26 in. deep. The concrete was proportioned from the Abrams water-cement ratio curve for a strength of 2,000 lb. at 28 days, water-cement ratio 1.00 by volume. The ingredients for one batch were: water 16 lb., cement 25 lb., sand (containing about 4 per cent water) 60 lb., enough ¾-in. gravel to make the mixture have a slump of 7 or 8 inches. The water in the sand caused the water-cement ratio to exceed 1.00 somewhat; it was expected that this water would either stay in the sand or its equivalent would be absorbed by the fairly dry gravel. It was known, also, that Abrams' curve is somewhat conservative for cements obtainable now. Test specimens made from this concrete bore this out, three 6 x 12-in. cylinders, cured for 28 days in damp sand, breaking at 2200, 3170, and 2550 lb. per square inch, an average of 2810. The test beam was cured in the forms.

The concrete for the beam was poured in the top of the form and allowed to run down on both sides of the top flange. Much puddling and rapping of the forms was resorted to, one man of the three spending all his time at this. Every effort was made to avoid honeycombing. The placing proceeded from one end toward the other, and the bottom form was rapped in an effort to cause the concrete to flow under the bottom flange from the end rather than from the sides. In spite of this care, after the test an air space as much as one inch thick was found under the center of the bottom flange all along except at the extreme ends. The appearance of the bottom was good. There was no indication of a hollow under the surface. Near the middle of the beam for a distance of about four feet a longitudinal wire of the mesh came about three-quarters of an inch above the edge of the bottom flange. Here the coarse aggregate wedged and the bottom corner of the concrete was not properly filled out. Hollow spaces were found under the top flange on both sides for the entire length except at the extreme ends.

It was intended that the test of Beam C should be as closely comparable as possible with that of Beam B to determine the relative values of gunite and concrete for encasement. Beam C was tested in the 400,000-lb. beam machine instead of in the adapted column machine, but this should not affect the results of the test.

Composite Beam C had a moment of inertia 2025 in.[4], section modulus for compression 190.5 in.[3], section modulus for tension 151.2 in.[3] The location of the neutral axis (computed by the

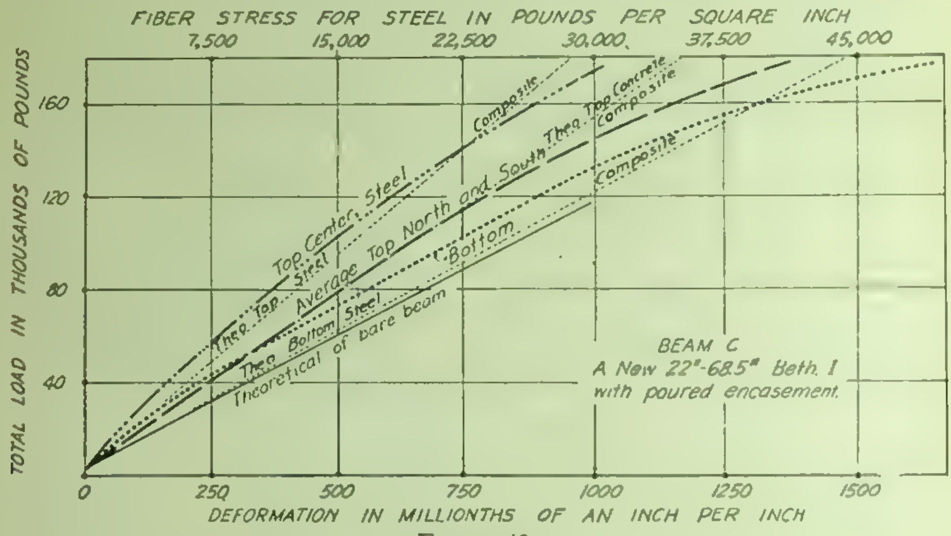


FIGURE 49

transformed section method on the straight-line principle and considering all steel, including the longitudinal wires of the mesh as working, and only that concrete above the neutral axis as carrying bending stress) was 10.62 in. down from the extreme top. The ratio of the moduli of elasticity was taken as 15.

Extensometer readings were taken on the steel at the center of the top and bottom flanges and on the concrete at both sides of the top flange. The drop of the scale beam was first noticed at a load of 81,800 lb.; it dropped in about 15 seconds at 121,800, in 3 seconds at 176,800 lb. A horizontal shear crack 5 or 6 in. below the top was noticed at 126,000 lb. The maximum load at failure was 202,630 lb.

Flange readings showed a tendency for the top to deflect toward the north and were slightly lower than those for Beam B. Figure 49 shows the Beam C load deformation curves. The lower curve is to be compared with the corresponding curve of Figure 48 for the gunite-encased beam. Beam C shows more load carried but

its section modulus is higher. The test curve for the bottom runs closer to the theoretical for Beam C than for Beam B. This might indicate that the poured concrete acted much less in tension than the gunite, in spite of there being so much more concrete below the neutral axis of Beam C than of Beam B. The "Top Center Steel" curve shows the test results above the theoretical indication for the composite beam, making the concrete beam appear more on the safe side in this respect than many of the gunite-encased

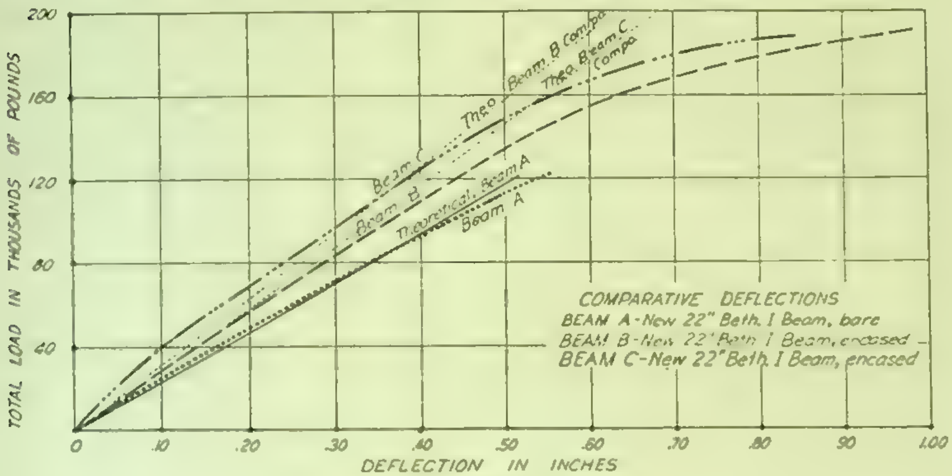


FIGURE 50

beams. Also Beam C was not perfectly poured; it was, however, as might be expected in many cases in actual practice.

This test shows that poured concrete may be expected to do its full share of reinforcing as well as of protecting.

Figure 50 shows the deflections of this group of beams. It will be noted that Beam C is as much stiffer than Beam B as Beam B is than the original steel beam. This is regardless of the serious fault of honeycombing in Beam C. Beam B is less stiff all along than its theoretical curve, whereas Beam C is stiffer up to heavy working loads. Using "n" as 15 in computation seems to be about right for the poured concrete, but "n" at 10 for the gunite seems to be much too low.

Tests of Beams B and C, the one encased in gunite and the other in poured concrete, suggest that stiffness does not follow strength when two such different materials are considered. Tests have shown gunite to be a high strength concrete, and the general features such as water-cement ratio and density indicate the same thing. The tests reported make it appear that 10 as a value of

"n" as for a 3,000-lb. concrete is too low. The column tests, which will be discussed later, denote the same thing.

Attempts were made to get the strength of the gunite. Prisms 4 x 4 x 8 in. were shot into forms from one side and were tested on end at 28 days. The results were not good, as it appeared impossible in placement in small, confined spaces, to avoid sand pockets which caused planes of varying degrees of weakness. One

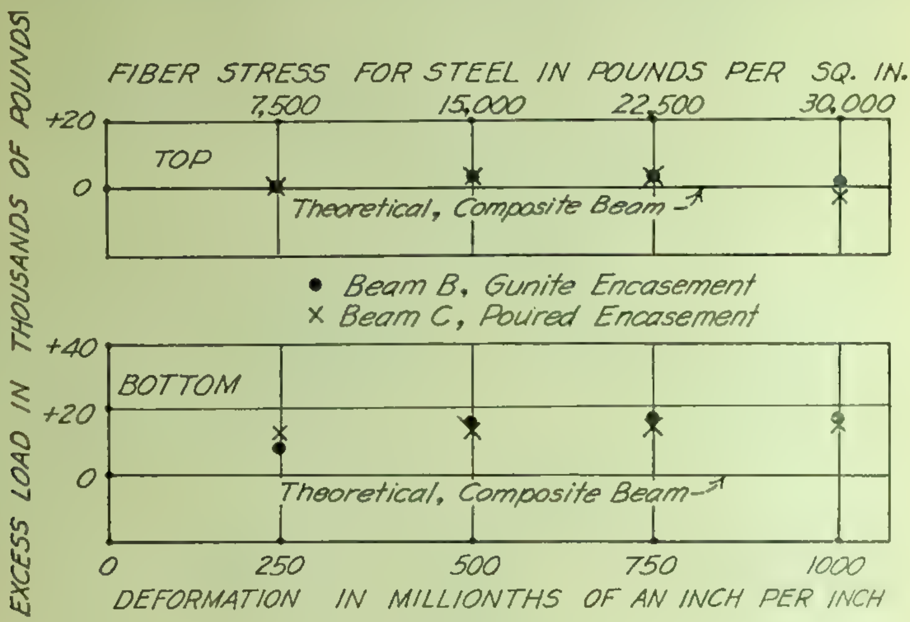


FIGURE 51

prism, the one showing the highest result, broke at a unit strength of 3160 lb. per square inch. Others broke at values down to 965 lb. per square inch. This last one showed two planes in line with the pressure. It appears that the only way to get a good test specimen is to core drill it from gunite built up on a flat vertical surface such as a girder web.

In placing gunite the operator should be careful in the use of the forms known as shooting strips not to get accumulations of sand due to secondary rebounding which leaves the sand in the gunite. All rebound sand should be free to fall completely off the work. The shooting strips used on some of the beams (they are shown in Figures 2 and 9) caused a few sand pockets which were later chipped out and reshot with new gunite made with the quick-setting alumina cement known as Lumnite. After setting and being broken down in the testing machine these gunite patches ap-

peared to make a perfect bond with the old gunite.* The change would not have been evident except for the color difference. The total amount of this patching did not exceed two cubic feet.

Figure 51 shows a comparison of Beams B and C on a basis of the theoretical curves for each, and is plotted similarly to Figures 45 and 46. It will be noticed that the bottom loads run about

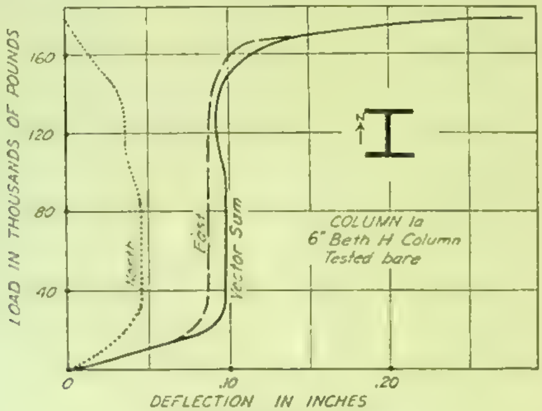


FIGURE 52

12,000 lb. over the theoretical in both beams at 15,000 and also at 22,500 lb. per square inch. This is about 19 per cent and 13 per cent respectively at these two points. The top values run along the theoretical values for both beams very closely to 22,500 lb. per square inch. The agreement of these two beams as to deformation is rather odd in the light of the comparative deflections.

TESTS OF COLUMNS

Bethlehem H columns, 6 in. by 20 lb., 18 ft. 10 in. long, slenderness ratio 150.7, were used in all these tests. Three columns were built out to 10 in. x 10 in. with gunite by the Fritz-Rumer-Cooke Company of Columbus at their yards and transported to the Engineering Experiment Station a day or two before they were tested. The gunite was portland cement and river sand in the ratio of about 1:2½. Welded wire mesh, 2 in. square, No. 12 wire both ways, was wrapped around the steel column. No longitudinal bars were used. Three of the columns were formed and concrete was poured around them to the same dimensions, 10 in. by 10 in. The same sort of mesh as for the gunited columns was used and in the same way. The concrete used for the poured encased columns

* Kennerley Bryan, Jr., reports that he inspected Beam B which had been exposed to weathering for more than a year after the tests and found the bond between Lumnite and ordinary cement gunite not nearly so good as it had appeared soon after encasement.

was the same mix as that used for Beam C. The entire length was formed and poured at one time. As the space between column and form was small for the length, puddling was out of the question, so reliance was placed in rapping the forms with a hand hammer. This proved to be sufficient as there was no honeycombing. Eight cylinders made from the same concrete were broken after 28 days' curing in damp sand. They averaged 2460 lb. per square inch with 2640 and 2130 lb. per square inch as extremes. In both the gunited and the poured columns the placing was good.

The gunite encased columns, 2a, 2b, and 2c, were tested at 63 days; poured columns, 3a and 3 b were tested at 64 days, and poured column 3c at 58 days. One column, No. 1a, was tested bare, and one of the gunited columns, 2c, after being tested encased was stripped and tested without encasement as 1b.

When the encasement was removed from the steel after testing it was found to require very much more work to remove the gunite than the poured concrete. After a large piece of gunite had been removed by means of wedges between the steel and concrete it was often necessary to chip off the remaining gunite which parted from the wedged block and remained adhering to the steel. This was not true of the poured concrete, though the latter was found to be in excellent contact.

The columns were tested in the 500,000-lb. column testing machine. The encasement for both poured concrete and gunite columns was 1 to 2 in. shorter than the length of the steel. Square ends were used without any ball joints and the load in all cases was applied to the ends of the steel column through hardened steel plates rigidly held in the testing machine. These ends had been planed by the rotary end planer at the works of the Mt. Vernon Bridge Company. Just prior to testing the ends were cleaned and lightly filed to remove small burrs that had been formed during handling.

Strain gage readings were taken on all four corners of the H at the column mid-height. The strain gage points were inserted at angles of 45 degrees with the webs and flanges. In all the encased columns it was necessary to cut holes through the encasement to the steel and to cut some of the gunite or concrete away for 20 in. The exposed places at the strain gage points were about one and one-half inches long. This cutting was much the same on all the columns. The side deflections were measured much in the same manner as were the vertical deflections of the beams. Plugs were placed on two sides of the column, 6 in. down from the top and 6 in. up from the bottom, at about the axes of the sides.

Fine hard-steel wires were stretched from top plug to bottom plug on each side. Each wire was tied to its top plug and was run over the bottom plug to another plug off the axis beyond which a free-hanging weight was attached. The lower two plugs on each side had polished surfaces over which the wires would slide easily as the column shortened.

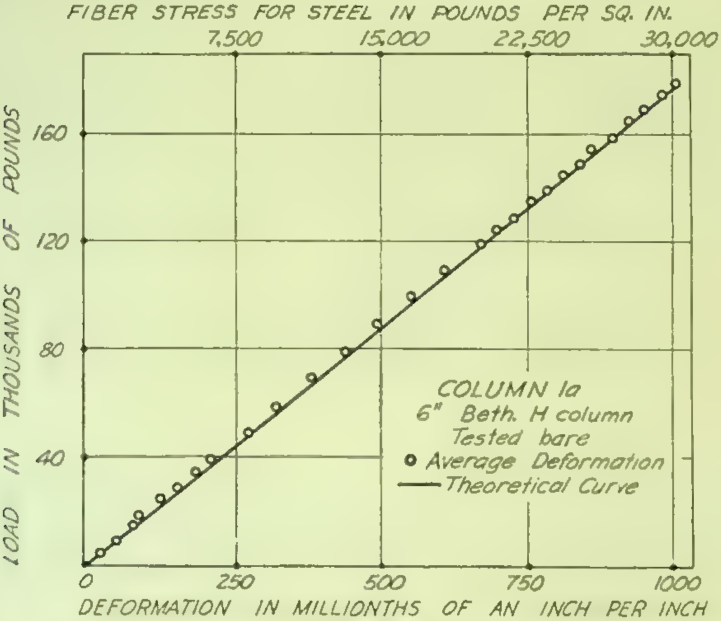


FIGURE 53

TESTS OF NAKED COLUMNS

Column 1a, tested bare, stood a maximum load of 180,000 lb., 377 lb. of which was the weight of the column itself. Figure 52 shows the deflection action of this column. Inaccuracies in the end bearings were adjusted in the first 30,000 lb., and the column contracted without bowing up to 130,000 lb., after which it began to buckle. At the end of the test the column showed points of inflection 2 or 3 ft. from each end. Loads were not carried beyond that producing a deflection of 3 or 4 in. The final bowing took place after the maximum load was reached, and its action was rapid enough to be plainly visible to the eye.

Figure 53 shows the load deformation curve of Column 1a as compared with the theoretical curve computed for the modulus of elasticity at 30,000,000. Each of these points is the average of the four strain gage readings. This column stood an average unit stress of 31,000 lb. per square inch just before failure.

Column 1b. After the test of encased Column 2c, in which the steel was not harmed in any way except for some upsetting at the ends, the gunite was cut off with hammer and chisel and the steel went through another test as Column 1b. Figure 54 shows how

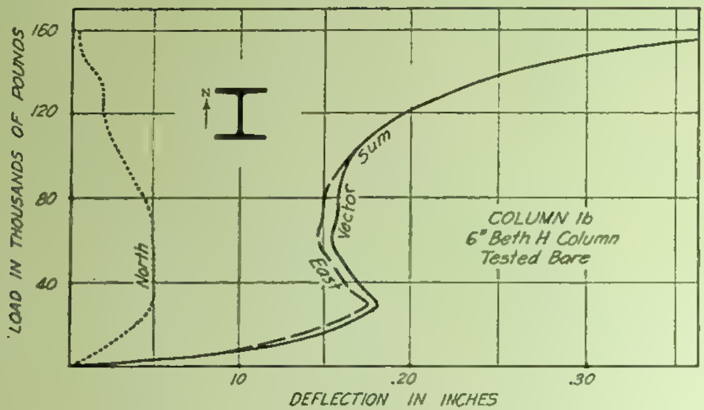


FIGURE 54

this column acted in the test. When the column was plumbed in the machine it was found possible to insert a piece of paper under one side of the end between the column and the hard steel plate of the testing machine. When the bottom was centered and in

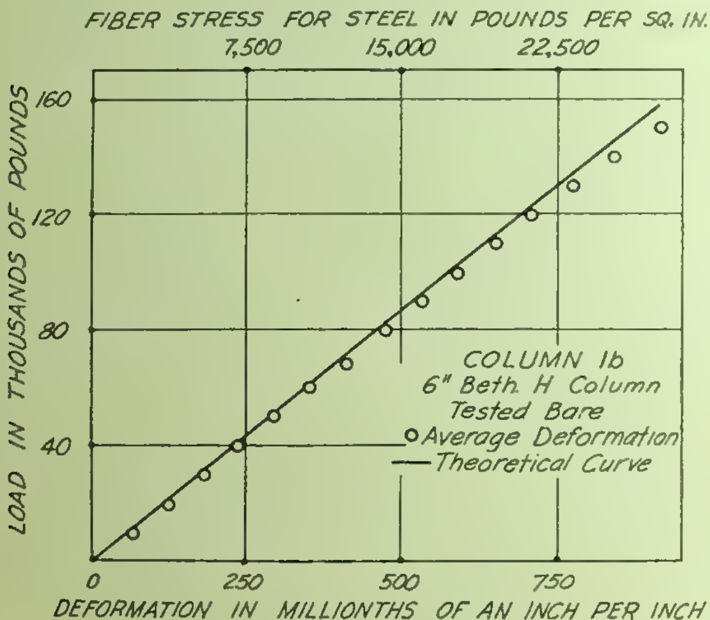


FIGURE 55

contact over the whole cross-section, the top was about one inch off center and leaning toward the west. The column was centered, both top and bottom, for the test. This column withstood a total load of 160,000 lb. as its maximum. This is an average of 27,500 lb. per square inch. Figure 55 shows the load deformation curve for Column 1b. It will be noticed that a line through the plotted points runs parallel to the theoretical curve up to a load of 120,000 lb. During the first 10,000 lb. of loading, adjustments at the ends threw these points off the theoretical curve.

Gunite Encased Columns

Figure 56 shows the lateral deflection data plotted for gunite-encased Column 2b. Column 2a had no appreciable deflection, and

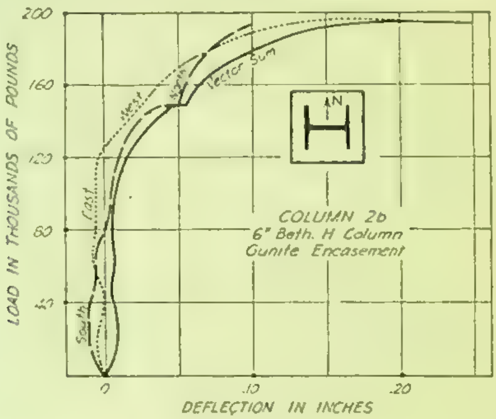


Figure 56

Column 2c bowed decidedly to the south, starting at the initial load and reaching the same amount at failure that it shown for Column 2b. There was a wide variation in the behavior of these three. Column 2b at 150,000 lb. showed the effect of holding the load for 5 minutes. During this pause there was a drop of 3,350 lb. in the load. The strain gage readings do not show any readjustment of the steel and gunite during this pause. A pause of 30 minutes was made at 130,000 lb. load for Column 2c. This does not show up in the lateral deflections, but is shown by some slight readjustment of the steel and gunite. The steel deformation showed a small increase during the pause, indicating some flow in the gunite. There was an 1800-lb. drop off in load during the 30-minute pause.

The maximum loads taken by Columns 2a, 2b, and 2c were, respectively, 221,600 lb., 206,750 lb., and 246,850 lb. In all cases the

scale beam began to fall during a pause in the loading at from 100,000 lb. to 150,000 lb., and this fall became more rapid as the loading advanced. In one case, 2c, a loss of 1,800 lb. was noted near the end of the test in the 2 minutes necessary to make a set of readings. In all cases the failure was due to upsetting of the steel at the ends of the column on account of its being above the elastic limit. This was evidenced by the gunite's being cracked 12 in. or more from the ends. At all other points on the column

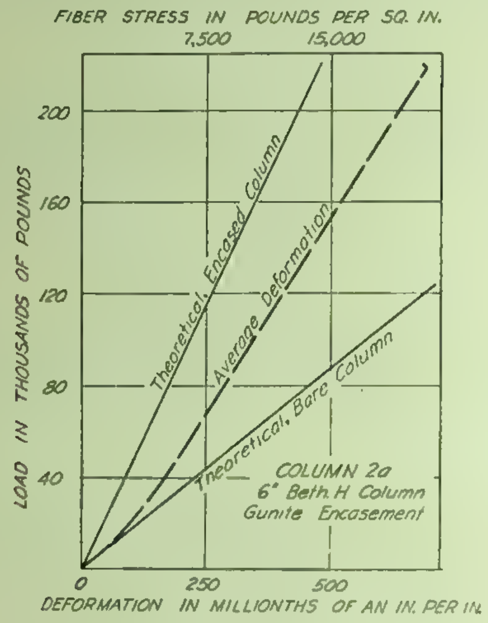


FIGURE 57

the gunite was unharmed and the bond was not broken. In the case of Columns 2b and 2c the pressure was continued for 5 minutes after the column would take no more load. The speed of the crosshead was 0.05 in. per minute. In none of these cases was there any marked bowing of the column.

Figures 57, 58, and 59 show the load deformation curves for Columns 2a, 2b, and 2c respectively. On these sheets also are plotted theoretical curves, one for the bare steel column based on its cross-section without taking into account any column action, and another for the steel and gunite combined, transforming gunite into its equivalent steel area by dividing by 10 as in the case of the beams. These test curves are quite similar in their appearance. Column 2a seems to have required a longer time for its initial adjustment, but its curve runs steeper than the other

two and meets them at about 150,000 lb. load. The curve for 2c shows the effect of the 30-minute pause at 130,000 lb. load. The

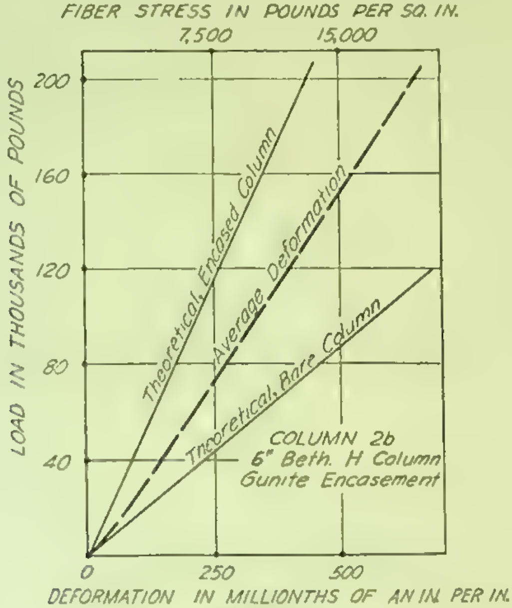


FIGURE 58

upper part of the curve is steeper and makes up for the loss before reaching the 170,000 lb. load. It will be noticed that the curves

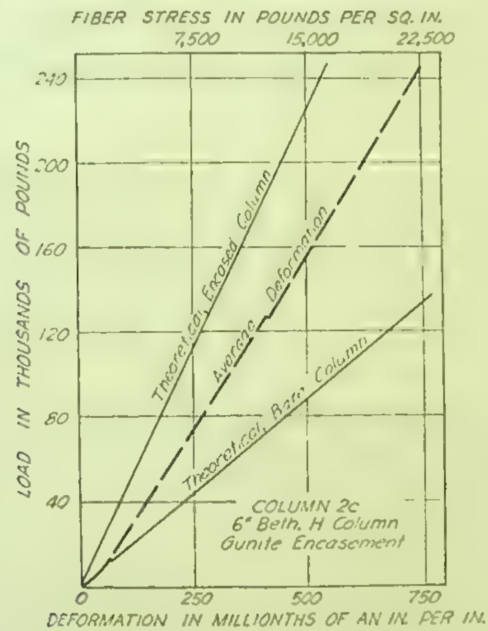


FIGURE 59

are considerably below that for the theoretical encased column. This theoretical curve should represent a case where the load would be applied to both steel and gunite. In this test the load was applied to steel only, and the gunite gave help only by way of bond. As an appreciable length at the ends was necessary to develop this bond, it is extreme to figure the whole length of the combined column as acting. This is what the "Theoretical Encased Column" curve represents.

Figure 17 shows the load deformation curve for the average of the three gunite-encased columns. At a load of 50,000 lb. which is about what the Bethlehem Steel Company advises as a safe working load for the bare steel, the steel is taking 62.8 per cent of all the load. At 90,00 lb. load where the steel would be at about 16,000 lb. per square inch stress if bare, 60.5 per cent of the load is carried by the steel. At 130,000 lb. load the steel carries 58.5 per cent. At these three points the values for "n" would be 27.4, 23.7, and 22.6 respectively. The percentages just given must be taken with considerable caution on account of the flow adjustment of the gunite. Just how much this might be over a term of days, months or years cannot be said at this stage of the investigation. The jump in the curve of Figure 59 indicates some flow adjustment for a period of 30 minutes. It is known that the flow is greater at early ages of loading of concrete.

Poured Encased Columns

Figure 60 shows the lateral deflection behavior of poured concrete encased Column 3a which deflected markedly to the south-

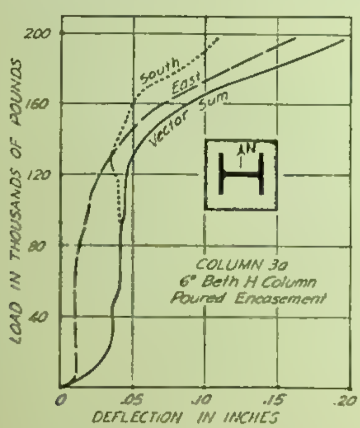


FIGURE 60

east above 120,000 lb. Columns 3b and 3c show no marked tendency to deflection anywhere in the test. Dropping of the scale

beam and final failure were about the same as for the gunite encased columns. The failures in all cases were by upsetting of the steel at the ends. Column 3a reached a maximum total load of 191,800 lb. Column 3b reached 248,000 lb. and 3c 241,400 lb. In all three cases the weight of the column was 2,100 lb. A 30-minute pause was made on Column 3c at a load of 130,000 lb., and during this pause there was a 2,900 lb. drop off of load.

Figures 61, 62 and 63 show the load deformation curves for

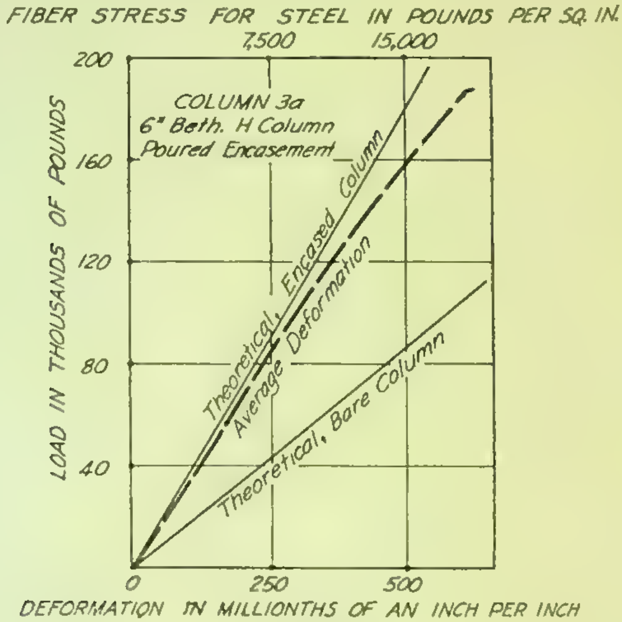


FIGURE 61

the three poured concrete encased columns. Figure 61 shows the test of Column 3a to be not very satisfactory; the curve drops off after the load of 120,000 lb. where the deflection begins, as shown on Figure 60. The concrete of Columns 3a and 3b was placed the same afternoon, the lower halves of both having been poured before the upper half of either. Figure 62 shows Column 3b to be very much the stiffer. This is the best test of the entire lot. It will be noted how much better this column tested than the theoretical curve based on "n" at 15. It is nearly as good as "n" at 10. Figure 63 shows a curve almost as good as Figure 62. It also shows the effect of the 30-minute pause. The comeback is very definite as in the case of the gunite-encased Column 2c, Figure 59.

Figure 18 shows the average load deformation curve for the three poured encased columns. This should be compared with

Figure 17 for the gunite-encased columns. By Figure 18, with 50,000 lb. on the scales the concrete takes 54.7 per cent and the

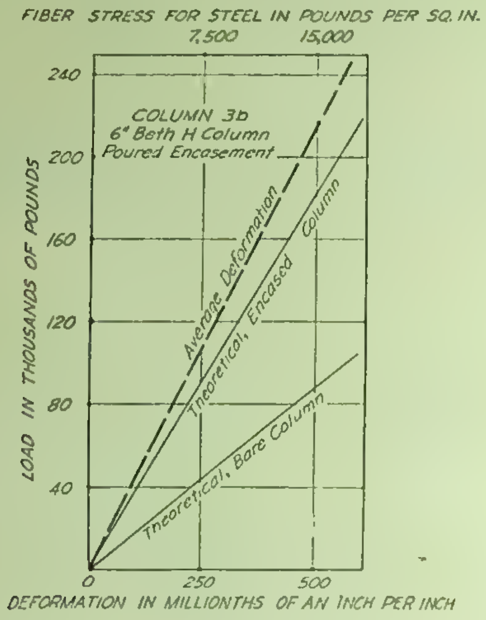


FIGURE 62

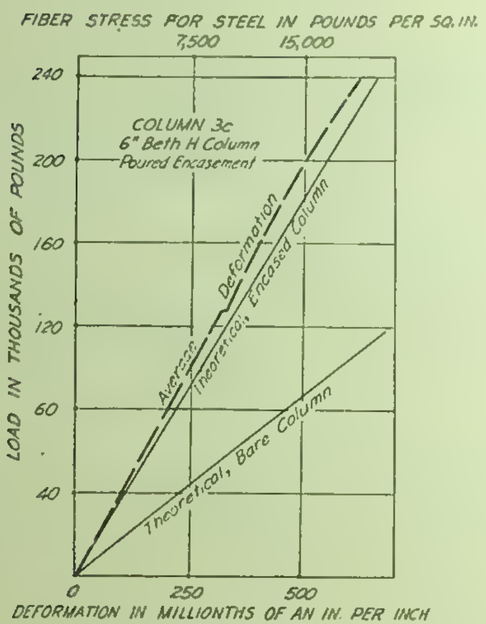


FIGURE 63

steel 45.3 per cent of the total load. At 90,000 lb. load these values are 54.5 per cent and 45.5 per cent respectively. At 130,000 lb. load they are 53.4 per cent and 46.6 per cent. At these loadings the values for "n" are 13.4, 13.5 and 14.2. The poured-concrete columns did not show the period of adjustment at the beginning of the loading that was noticeable for the gunite columns.

APPENDIX

CALCULATION METHODS

In order for the tests to have meaning it was necessary that observed loads and deformations should be compared with calculated or predicted values. As explained on page 4 the transformed section method was used in computing the properties of the test specimens. Figure 3 is a typical composite beam section showing certain dimensions used in computation.

A_b, Area of the steel beam

I_b, Moment of inertia of steel beam

A_s, Area of steel reinforcing bars

n, Ratio of modulus of elasticity of steel to that of concrete

b, Effective width of slab which may be considered as flange.

I, Equivalent moment of inertia of transformed section

f_c, Extreme fiber stress in concrete at the top

f^1_{sb}, Extreme fiber stress in top of steel beam

f_s, Unit stress in steel reinforcing rods

f_{sb}, Extreme fiber stress in bottom of steel beam

The relations between these various quantities are expressed by the formulas

$$kd = \frac{bt^2 + 2n\,(A_b a + A_s d)}{2[bt + n\,(A_b + A_s)]}$$

$$I = bt\left[\frac{t^2}{12} + \left(kd - \frac{t}{2}\right)^2\right] + n[I_b + A_b\,(a - kd)^2 + A_s(d - kd)^2]$$

$$f_c = \frac{M\,kd}{I}$$

$$f^1_{sb} = \frac{M\,(kd - d^1)}{I}$$

$$f_s = \frac{M\,(d - kd)n}{I}$$

$$f_{sb} = \frac{M\,(h - kd)n}{I}$$

The method of calculating may be illustrated by this problem: Suppose the stringers of a highway bridge are 24-in. I-beams, 20 ft. long, which have rusted until the minimum section has the following properties:

Area of cross section 10.15 sq. in.

Eccentricity of neutral axis 2.05 in. (above center)

Moment of Inertia 1009 in.^{4}

Assume that the deck weighs 700 lb. per linear foot of beam. The resulting dead load moment is 35,000 foot-pounds.

For live load assume a 20-ton truck with 80 per cent of the load on the rear axle and 25 per cent impact. The maximum live-load-plus-impact moment is 100,000 foot-pounds.

The calculated maximum stress for the unreinforced section would be

DL =	5850 lb. per square inch
LL =	16700 lb. per square inch
Total	22550 lb. per square inch

Assume that the deck is to be undisturbed and that the stringers are to be reinforced and encased to the section shown in Figure 38. The encased beam will weigh approximately 200 lb. per linear foot, making a total dead load of 900 lb. per foot of beam and a dead load moment of 45,000 foot-pounds. This dead load moment gives the following maximum extreme fiber stresses in the stringer.

Top flange	5375 lb. per square inch
Bottom flange	7520 lb. per square inch

The kd for the composite beam is 10.88 in. and the moment of inertia 26205 in.^{4} With live load moment remaining 100,000 foot-pounds

$$\text{LL } f_c = \frac{100{,}000 \times 12 \times 10.88}{26205} = 498 \text{ lb. per square inch}$$

$$\text{LL } f_s = \frac{100{,}000 \times 12 \times 14.37}{26205} \times 15 = 9855 \text{ lb. per square inch}$$

$$\text{LL } f_{sb} = \frac{100{,}000 \times 12 \times 13.12}{26205} \times 15 = 9000 \text{ lb. per square inch}$$

This gives for the composite beam a maximum stress in the bottom flange of the old stringer of

$$\text{DL } f_{sb} = 7520$$
$$\text{LL } f_{sb} = 9000$$

Total 16520 lb. per square inch

In calculating the safe load for the composite section, it should be considered safe to allow the total fiber stress in the old steel to run as high as 24,000 lb. per square inch so long as the usual fiber stresses of 750 lb. per square inch in the concrete and 16,000 lb. per square inch in the reinforcing steel are not exceeded, because the ultimate strength of the composite beam is not reached until the new steel or concrete fails.

TABLE NO. IV

Caliper Notes of Corroded Plate Girders

BEAM XI

Section at Center		Area Sq. Ins.	Dist. from Center Line	Statical Moment	Moment of Inertia	
Web	21 x 3/8	7.87			289.41	
4 Vert. Legs	3 x 3/8	4.50			367.85	
Marked Flange Angles						
	3—1/8 x 5/16	0.59	10.41	6.1419	63.94	
	3—1/8 x 1/8	0.39	10.44	4.0716	42.51	
Plain Flange Angles						
	3—1/8 x 5/16	0.98	—10.34	—10.1332	104.78	
	3—1/8 x 1/4	0.78	—10.37	— 8.0886	83.88	952.37
		15.11	— 0.53	— 8.0083		4.24
						948.13 in⁴
Marked Flange in Tension						
Gross Section		15.00	— 0.53	— 8.0083	+952.37	
Less 1 hole	13/16 x 3/8	—0.30	+ 8.75	— 2.6250	— 22.97	
2	0.71 x 3/8	—0.53	+ 8.75	— 4.6375	— 40.58	
1	0.71 x 3/16	—0.13	+10.41	— 1.3533	— 14.08	
1	0.71 x 1/8	—0.09	+10.44	— 0.9396	— 9.81	
1	0.81 x 5/16	—0.25	—10.34	+ 2.5850	— 26.73	
1	0.81 x 1/4	—0.20	—10.37	+ 2.0740	— 21.51	816.69
		13.61	— 0.95	—12.9047		12.26
						804.43 in⁴
Marked Flange in Compression						
Less 1 hole	13/16 x 3/8	—0.30	— 8.75	+2.6250	—22.97	
2	0.71 x 3/8	—0.53	— 8.75	+4.6375	—40.58	
1	0.71 x 5/16	—0.22	—10.34	+2.2748	—23.52	
1	0.71 x1/4	—0.18	—10.37	+1.8666	—19.36	
1	0.81 x 3/16	—0.15	+10.41	+1.5615	—16.26	
1	0.81 x 1/8	—0.10	+10.44	—1.0440	—10.90	818.78
		13.63	+ 0.06	+0.7901		0.05
						818.73 in⁴

Section Moduli at Center

Gross Section	Marked Flg.	948.13 ÷ 11.03 = 85.96 in³	69%
	Plain Flg.	÷ 9.77 = 95.10	77%
Marked Flg. in Compression	Marked Flg.	818.73 ÷ 10.44 = 78.42	73%
	Plain Flg.	÷ 10.56 = 77.53	79%
Marked Flg. in Tension	Marked Flg.	804.43 ÷ 11.45 = 70.26	71%
	Plain Flg.	÷ 9.55 = 84.23	78%

BEAM XII

Section at Center		Area Sq. Ins.	Dist. from Center Line	Statical Moment	Moment of Inertia	
Web	21 x 3/8	7.87			289.41	
4 Vert. Legs	3 x 3/8	4.50			367.85	
Marked Flange Angles						
	3—1/8 x 5/16	0.98	10.34	10.1332	104.78	
	3—1/8 x 5/16	0.98	10.34	10.1332	104.78	
Plain Flange Angles						
	3—1/8 x 1/4	0.78	—10.37	— 8.0886	83.88	
	3—1/8 x 5/16	0.98	—10.34	—10.1332	104.78	1055.48
		16.09	+ 0.13	+ 2.0446		0.27
						1055.21 in ^{4}

Marked Flange in Compression						
Less 1 hole	13/16 x 3/8	—0.30	— 8.75	+2.6250	—22.97	
2	0.71 x 3/8	—0.53	— 8.75	+5.6375	—40.58	
1	0.71 x 1/4	—0.18	—10.37	+1.8666	—19.36	
1	0.71 x 5/16	—0.22	—10.34	+2.2748	—23.52	
1	0.81 x 5/16	—0.25	+10.34	—2.5850	—26.73	
1	0.81 x 5/16	—0.25	+10.34	—2.5850	—26.73	895.59
		14.36	+ 0.58	+8.2785		4.80
						890.79 in ^{4}

Marked Flange in Tension						
Gross Section		+16.09	+ 0.13	+2.0446	+1055.21	
Less 1 hole	13/16 x 3/8	— 0.30	+ 8.75	—2.6250	— 22.97	
2	0.71 x 3/8	— 0.53	+ 8.75	—4.6375	— 40.58	
1	0.71 x 5/16	— 0.22	+10.34	—2.2748	— 23.52	
1	0.71 x 5/16	— 0.22	+10.34	—2.2748	— 23.52	
1	0.81 x 1/4	— 0.20	—10.37	+2.0740	— 21.51	
1	0.81 x 5/6	— 0.25	—10.34	+2.5850	— 26.73	896.38
		14.37	— 0.36	—5.1077		1.84
						894.54 in ^{4}

Section Moduli at Center

Gross Section	Marked Flg.	1055.21 ÷ 10.37 = 101.76 in ^{3}	82%
	Plain Flg.	÷ 10.63 = 99.27	80%
Marked Flg. in Compression	Marked Flg.	890.79 ÷ 9.92 = 89.80	83%
	Plain Flg.	÷ 11.08 = 80.40	81%
Marked Flg. in Tension	Marked Flg.	894.54 ÷ 10.86 = 82.37	83%
	Plain Flg.	÷ 10.14 = 88.22	82%

BEAM XIII

Section at Center		Area Sq. Ins.	Dist. from Center Line	Statical Moment	Moment of Inertia	
Web	21 x 3/8	7.87			289.41	
4 Vert. Legs	3 x 3/8	4.50			367.85	
Marked Flange Angles						
	3—1/8 x 15/64	0.73	10.38	7.5774	78.65	
	3—1/8 x 1/8	0.39	10.44	4.0716	42.51	
Plain Flange Angles						
	3—1/8 x 3/16	0.59	—10.41	—6.1419	63.94	
	3—1/8 x 5/32	0.49	—10.42	—5.1058	53.20	895.56
		14.57	+ 0.03	+0.4013		.01
						895.55 in ⁴

Marked Flange in Compression

Less 1 hole	13/16 x 3/8	—0.30	— 8.75	+2.6250	—22.97	
2	0.71 x 3/8	—0.53	— 8.75	+4.6375	—40.58	
1	0.71 x 3/16	—0.13	—10.41	+1.3533	—14.08	
1	0.71 x 5/32	—0.11	—10.42	+1.1462	—11.94	
1	0.81 x 15/64	—0.19	+10.38	—1.9722	—20.47	
1	0.81x1/8	—0.10	+10.44	—1.0440	—10.90	774.62
		13.21	— 0.54	+7.1471		3.86
						770.76 in ⁴

Marked Flange in Tension

Gross Section		+14.57		+0.4013	+895.56	
Less 1 hole	13/16 x 3/8	— 0.30	+ 8.75	—2.6250	— 22.97	
2	0.71 x 3/8	— 0.53	+ 8.75	—4.6375	— 40.58	
1	0.71 x 15/64	— 0.17	+10.38	—1.7646	— 18.32	
1	0.71 x 1/8	— 0.09	+10.44	—0.9396	— 9.81	
1	0.81 x 3/16	— 0.15	—10.41	+1.5615	— 16.26	
	0.81 x 5/32	— 0.13	—10.42	+1.3546	— 14.11	773.51
		13.20	— 0.50	—6.6493		3.32
						770.19 in ⁴

Section Moduli at Center

Gross Section	Marked Flg.	895.55 ÷ 10.47 = 85.53 in ³	69%
	Plain Flg.	÷ 10.53 = 85.05	69%
Marked Flg. in Compression	Marked Flg.	770.76 ÷ 9.96 = 77.39	72%
	Plain Flg.	÷ 11.04 = 69.82	71%
Marked Flg. in Tension	Marked Flg.	770.19 ÷ 11.00 = 70.02	71%
	Plain Flg.	÷ 10.00 = 77.02	72%

BEAM XIV

Section at Center		Area Sq. Ins.	Dist. from Center Line	Statical Moment	Moment of Inertia	
Web	21 x 3/8	7.87			289.41	
4 Vert. Legs	3 x 3/8	4.50			367.85	
Marked Flange Angles						
	3—1/8 x 3/16	.59	10.41	6.1419	63.94	
	3—1/8 x 7/32	.68	10.39	7.0652	73.41	
Plain Flange Angles						
	3—1/8 x 5/16	.98	—10.34	—10.1332	104.78	
	3—1/8 x 5/16	.98	—10.34	—10.1332	104.78	1004.17
		15.60	— 0.45	— 7.0593		3.18
						1000.99 in^4

Marked Flange in Tension						
Gross Section		+15.60	— 0.45	— 7.0593	+1004.17	
Less 1 hole	13/16 x 3/8	— 0.30	+ 8.75	— 2.6250	— 22.97	
2	0.71 x 3/8	— 0.53	+ 8.75	— 4.6375	— 40.58	
1	0.71 x 3/16	— 0.13	+10.41	— 1.3533	— 14.08	
1	0.71 x 7/32	— 0.16	+10.39	— 1.6624	— 17.27	
1	0.81 x 5/16	— 0.25	—10.34	+ 2.5850	— 26.73	
1	0.81 x 5/16	— 0.25	—10.34	+ 2.5850	— 26.73	855.81
		13.98	— 0.87	—12.1675		10.59
						845.22 in^4

Marked Flange in Compression						
Less 1 hole	13/16 x 3/8	—0.30	— 8.75	+2.6250	—22.97	
2	0.71 x 3/8	—0.53	— 8.75	+4.6375	—40.58	
1	0.71 x 5/16	—0.22	—10.34	+2.2748	—23.52	
1	0.71 x 5/16	—0.22	—10.34	+2.2748	—23.52	
1	0.81 x 3/16	—0.15	+10.41	—1.3615	—16.26	
1	0.81 x 7/32	—0.18	+10.39	—1.8702	—19.43	857.89
		14.00	+ 0.09	+1.3211		0.12
						857.77 in^4

Section Moduli at Center

Gross Section	Marked Flg.	1000.99 ÷ 10.95 = 91.41	73%
	Plain Flg.	÷ 10.05 = 99.60	80%
Marked Flg. in Compression	Marked Flg.	857.77 ÷ 10.41 = 82.40	77%
	Plain Flg.	÷ 10.59 = 81.00	82%
Marked Flg. in Tension	Marked Flg.	845.22 ÷ 11.37 = 74.34	75%
	Plain Flg.	÷ 9.63 = 87.77	82%

TABLE NO. V

CALIPER NOTES OF CORRODED I-BEAMS

BEAM XV

Section at Center		Area Sq. Ins.	Dist. from Center Line	Statical Moment	Moment of Inertia	
Web	24 x 3/16	4.50			216.00	
Marked						
Flg.	2—7/8 x 0					
	1/2 x 2—7/8 x 13/16	1.17	11.73	13.7241	160.98	
	2—7/8 x 1/8	0.36	11.94	4.2984	51.32	
	1/2 x 2—7/8 x 11/16	0.99	11.65	11.5335	134.37	
Plain						
	Flg.3—3/32 x 5/16	0.97	—11.84	—11.4848	135.98	
	1/2 x 3—3/32 x 11/16	1.06	—11.46	—12.1476	139.21	
	3—3/32 x 11/32	1.06	—11.83	—12.5398	148.35	
	1/2 x 3—3/32 x 21/32	1.02	—11.44	—11.6688	133.49	1119.70
		11.13	— 1.64	—18.2850		29.99
						1089.71 in⁴
						52%

Section Moduli at Center

Marked Flange 1089.71 ÷ 13.64 = 79.89 in³ 46%

Plain Flange ÷ 10.36 = 105.18 in³ 61%

BEAM XVI

Section at Center		Area Sq. Ins.	Dist. from Center Line	Statical Moment	Moment of Inertia	
Web	24 x 3/32	2.25			108.00	
Marked						
	Flg.2—33/64x7/32	0.55	11.89	6.5395	77.75	
	1/2 x 2—33/64 x 17/32	0.67	11.60	7.7720	90.16	
	2—33/64 x 0					
	1/2 x 2—33/64 x 13/16	1.02	11.73	11.9646	140.34	
Plain						
	Flg.3—7/64 x 3/8	1.17	—11.81	—13.8177	163.19	
	1/2 x 3—7/64 x 9/16	0.87	—11.44	— 9.9528	113.86	
	3—7/64 x 1/4	0.78	—11.88	— 9.2684	110.08	
	1/2 x 3—7/64 x 5/8	0.97	—11.54	—11.1938	129.18	932.56
		8.28	— 2.17	—17.9546		38.96
						893.60 in⁴
						43%

Section Moduli at Center

Marked Flange 893.60 ÷ 14.17 = 63.06 in³ 36%

Plain Flange ÷ 9.83 = 90.91 in³ 52%

BEAM XVII

Section at Center		Area Sq. Ins.	Dist. from Center Line	Statical Moment	Moment of Inertia	
Web	24 x 5/32	3.75			180.00	
Marked						
	Flg.2—53/64 x 0					
	1/2 x 2—53/64 x 13/16	1.15	11.73	13.4895	158.23	
	2—53/64 x 0					
	1/2 x 2—53/64 x 13/16	1.15	11.73	13.4895	158.23	
Plain						
	Flg.3—21/64 x 5/16	1.04	—11.84	—12.3136	145.79	
	1/2 x 3—21/64 x 11/16	1.14	—11.46	—13.0644	149.72	
	3—21/64 x 1/4	0.83	—11.88	— 9.8604	114.77	
	1/2 x 3—21/64 x 21/32	1.09	—11.53	—12.5677	144.91	1051.65
		10.15	— 2.05	—20.8271		42.70
						1008.95 in ^{4}
						48%

Section Moduli at Center

Marked Flange $1008.95 \div 14.05 = 71.81$ in ^{3} 41%

Plain Flange $\div 9.95 = 101.40$ in ^{3} 58%

BEAM XVIII

Section at Center		Area Sq. Ins.	Dist. from Center Line	Statical Moment	Moment of Inertia	
Web	24 x 9/32	6.75			324.00	
Marked						
	Flg.3—1/64 x 9/32	0.85	11.86	10.0810	119.56	
	1/2 x 3—1/64 x 21/32	0.99	11.50	11.3850	130.93	
	3—1/64 x 5/16	0.94	11.84	11.1296	131.77	
	1/2 x 3—1/64 x 5/8	0.94	11.48	10.7912	123.88	
Plain						
	Flg. 2—53/64 x 0					
	1/2 x 2—53/64 x 13/16	1.15	—11.73	—13.4895	158.23	
	2—53/64 x 0					
	1/2 x 2—53/64 x 27/32	1.19	—11.72	—13.9468	163.46	1151.83
		12.81	+ 1.25	+15.9505		19.94
						1131.89 in ^{4}
						54%

Section Moduli at Center

Marked Flange $1131.89 \div 10.75 = 105.29$ in ^{3} 61%

Plain Flange $\div 13.25 = 85.43$ in ^{3} 49%

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